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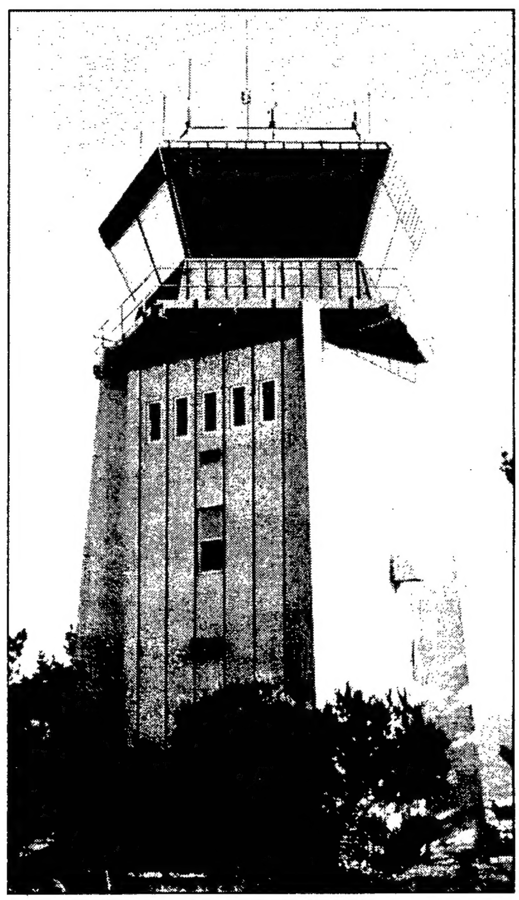
Seismic Evaluation of the Type L and San Luis Obispo Braced Frame FAA Airport Traffic Control Towers

James Wilcoski, Ernest Heymsfield, James Horne, George Manning, and Matthew Walters

Executive Order (EO) 12941 requires all Federal agencies to develop and submit "seismic upgrade" cost estimates to the Federal Emergency Management Agency no later than 1 December 1998. The U.S. Army Corps of Engineers assisted the Federal Aviation Administration (FAA) in responding to this EO by evaluating the seismic resistance of many of their facilities.

This report presents a detailed seismic evaluation of the FAA's Airport Traffic Control Towers in Salinas (shown at right), San Carlos, Palo Alto, and San Luis Obispo, CA.



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Executive Summary

This report presents a detailed seismic evaluation of the FAA Airport Traffic Control Towers (ATCTs) located in Palo Alto, Salinas, San Carlos and San Luis Obispo, CA. The San Luis Obispo tower is a unique, eccentrically braced steel frame tower. The three other towers are Type L towers, which are reinforced concrete-frame structures. Each was evaluated based on the maximum considered earthquake defined by 1997 National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions (FEMA 302). Type L and the San Luis Obispo towers were evaluated based on several directions of loading and an extreme assumption that the cab windows do not fail and work as fully effective shear walls.

Type L ATCTs

The San Carlos ATCT was the most critical Type L tower, due to excessive deflections in the tower cab. These deflections were due to large rotations of supporting members at the shaft roof. The cab columns (corner mullions) connection base plates were also overstressed as indicated by very large demand capacity ratios. Hinges would form at the base of each mullion due to base plate bending failure, causing a collapse mechanism at very low seismic motions.

An upgrade approach was developed and demonstrated that reduces deflections to acceptable levels and protects the vulnerable connections. This upgrade consists of welding deep structural tubing members to the base of each corner mullion in a pentagon configuration as shown in Figure 11 of the report. The mullions themselves were also stiffened and strengthened by welding 5" x 1.5" plates on both faces of the mullions.

San Luis Obispo ATCT

The shaft braces were the most critically stressed components in the San Luis Obispo tower shaft. These would buckle at several floor levels. However, when the braces were assumed to be tension-only members, they had adequate capac-

ity. These braces could yield slightly, but this would be very limited and deflections would be kept within acceptable levels.

Deflection in the tower cab could be large, but within acceptable levels. The most vulnerable cab component is the connection of the corner mullions to their base plates. This vulnerability is due to shear failure of the fillet welds at this connection. However, serious damage to these connections should be prevented by redistribution of forces to other mullions and other building components. Therefore, the San Luis Obispo tower passed this evaluation by meeting the life-safety requirements.

Foreword

This study was conducted for the Northwestern Division (NWD) of the U.S. Army Corps of Engineers under Military Interdepartmental Purchase Request (MIPR) W81THP80338948, for which the scope of work was defined by the U.S. Army Construction Engineering Research Laboratories (USACERL) proposal "Development of Analytical Procedures and Seismic Evaluation of Selected FAA Airport Traffic Control Towers (ATCTs)." The technical monitor was Bruce H. McCracken, CENWD-NP-ET-E.

This study supported a larger project including other Corps offices, to provide the Federal Aviation Administration (FAA) with the information it needs to formulate a long-range plan to provide seismic upgrades to its most critical facilities. This overall effort is defined in the Interagency Agreement Between the Department of Transportation/Federal Aviation Administration and the US Army Engineer Waterways Experiment Station on behalf of the Corps of Engineers. This work was in direct support of Executive Order (EO) 12941, "Seismic Evaluations of Existing Federally owned or Leased Buildings," dated December 1, 1994; which requires all Federal agencies to develop and submit "seismic upgrade" cost estimates with supporting documentation to the Federal Emergency Management Agency (FEMA) not later than December 1, 1998.

The work was performed by the Engineering Division (FL-E) of the Facilities Technology Laboratory (FL), USACERL. The principal investigator for this project was James Wilcoski. Ernest Heymsfield is an assistant professor for the Department of Civil Engineering at Louisiana State University. Larry M. Windingland is Chief, FL-E, and Michael Golish is Operations Chief, FL. The USACERL technical editors were Gordon L. Cohen and Linda L. Wheatley, Technical Information Team.

Dr. Michael J. O'Connor is the Director of USACERL.

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1 Introduction

Background

This work was conducted in response to Executive Order (EO) 12941, which promulgated the National Earthquake Hazards Reduction Program (NEHRP) Act, Public Law 101-614. EO 12941, "Seismic Evaluations of Existing Federally owned or Leased Buildings," dated December 1, 1994, requires all Federal agencies to develop and submit "seismic upgrade" cost estimates with supporting documentation to the Federal Emergency Management Agency (FEMA) not later than 1 December 1998.

The U.S. Army Corps of Engineers assisted the Federal Aviation Administration (FAA) in responding to this EO by evaluating the seismic resistance of many of their facilities. The Northwestern Division (NWD), Waterways Experiment Station (WES), and Construction Engineering Research Laboratories (USACERL) worked together to deliver these evaluations. The first phase of the evaluations included structural evaluation and guidance for nonstructural evaluation of FAA airport traffic control towers (ATCTs). The Corps provided detailed retrofit guidance and cost estimates for seismically vulnerable ATCTs. These evaluations were completed in 1996 for standard control towers in Seismic Zones 3 and 4.

The second phase of this work began in 1997, when the Corps evaluated numerous other FAA facilities.

Objective

The objective of this project was to conduct life-safety seismic evaluations of the FAA ATCTs at the California locations shown in Table 1. These structural evaluations included both the tower shaft and cabs. Retrofit schemes were also developed for towers found to be vulnerable.

Approach

This report presents the evaluation of the Type L concrete frame towers and a braced steel frame tower in Seismic Zone 4. Table 1 is a summary of the Type L and San Luis Obispo steel braced frame ATCTs taken from the URS Greiner final report.*

Figure 1 shows the Salinas tower, which will have the most critically loaded shaft of all the Type L towers because it has the greatest height (50 ft). Some interior members in the shorter San Carlos tower (Figure 2) may be loaded more severely than Salinas, because of the higher spectral acceleration levels. Therefore, selected members and connections will be evaluated based on the analysis of the shorter San Carlos tower. The Palo Alto and San Carlos towers are structurally identical, except that the San Carlos tower experiences greater seismic loads. Retrofit schemes derived for the San Carlos tower, therefore, would be sufficient for the Palo Alto tower.

The San Luis Obispo tower is a unique tower with a steel braced frame shaft and steel moment frame cab. Figure 3 shows the San Luis Obispo tower. All towers were evaluated for both gravity and seismic loading using SAP 2000 Finite Element Method software.†

Table 1. Summary of Type L and San Luis Obispo ATCTs.

California Location	Height at Cab Base (ft)	Year Built	UBC Seismic Zone	FEMA 302 Short Period Spectral Acceleration, S_{DS}	Construction Type	
					Shaft	Cab
Salinas (SNS)	50	1968	4	1.00 g	Reinforced concrete frame	Steel moment frame
San Carlos (SQL)	30	1969	4	1.07 g	Reinforced concrete frame	Steel moment frame
Palo Alto (PAQ)	30	1965	4	1.00 g	Reinforced concrete frame	Steel moment frame
San Luis Obispo (SBP)	45	1988	4	1.00 g	Steel braced frame	Steel moment frame

* URS Greiner, Federal Aviation Administration Categorization of Airport Traffic Control Towers (ATCTs) Seismic Screening of Terminal Radar Approach Control (TRACON) Buildings, for the U.S. Army Corps of Engineers, 20 May 1998.

† SAP2000 Structural Analysis Programs from Computers and Structures, Inc., Berkeley, CA.

The ATCTs shown in Table 1 were evaluated following the steps presented in Chapter 3, **Analysis Steps and Cases**.

Mode of Technology Transfer

This report will be incorporated into the overall Corps of Engineers effort to provide the FAA with the information it needs to formulate a long-range plan to provide seismic upgrades to its most critical facilities. This report will also contribute to the FAA response to EO 12941 requiring "Seismic Evaluations of Existing Federally Owned and Leased Buildings."

Figure 1. Salinas, CA, Type L ATCT, 50-ft tall.


Figure 2. San Carlos, CA, Type L ATCT, 30-ft tall.

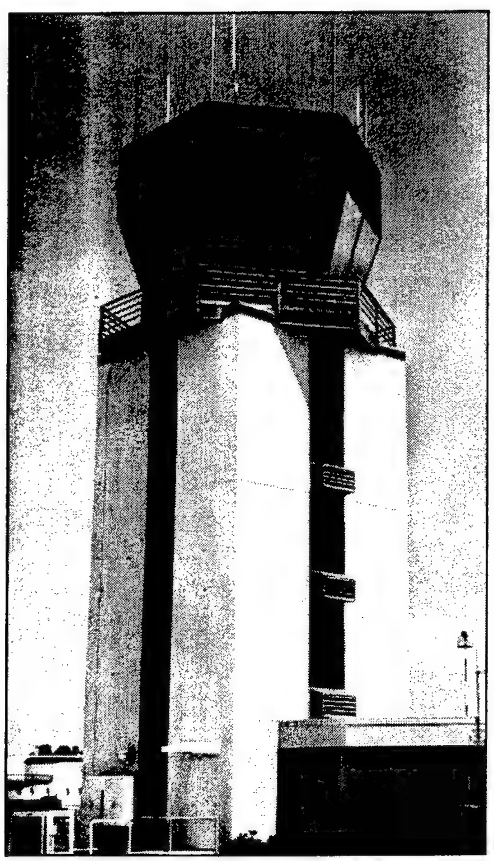
Figure 3. San Luis Obispo, CA, steel braced frame ATCT, 45-ft tall.

References

The following references were used in these evaluations:

1991 Edition of the NEHRP Recommended Provisions and Commentary for the Development of Seismic Regulations for New Buildings (FEMA 222 and 223), January 1992.

1997 Edition of the NEHRP Recommended Provisions and Commentary for Seismic Regulations for New Buildings and Other Structures (FEMA 302 and 303), February 1998.

1997 Uniform Building Code, Volume 2: Structural Engineering Design Provisions.

AISC Manual of Steel Construction, Load & Resistance Factor Design (LRFD), 2nd Edition, 1994.

Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95), American Concrete Institute, October 1995.

Corlett & Spackman, Drawings of Air Traffic Control Towers, San Carlos and Salinas, CA, (Corlett & Spackman, Architects, San Francisco, CA, June 1966).

Leo A Daly, Drawings of Administrative Base Building, San Luis Obispo, CA (Leo A Daly Architecture/Engineering, Omaha, NE, December 1981).

National Earthquake Hazards Reduction Program (NEHRP) Guidelines (FEMA 273) and Commentary (FEMA 274) for the Seismic Rehabilitation of Buildings, October 1997.

Units of Weight and Measure

U.S. standard units of measure are used throughout this report. A table of conversion factors for Standard International (SI) units is provided below.

SI conversion factors		
1 in.	=	2.54 cm
1 ft	=	0.305 m
1 kip force	=	4.45 kN
1 k-in.	=	113 N-m
1 ksi	=	6.89 MPa
1 lb mass	=	0.453 kg
1 lb/ft	=	1.488 kg/m
1 pcf	=	16.02 kg.m^3
1 psf	=	4.882 kg/m^2

2 Tower Configurations, Model Assumptions, and Load Conditions

The analytical approach used included gravity and response spectrum analysis, first using a linear dynamic procedure (LDP; FEMA 273, 3.3.2) and then if needed using a nonlinear static procedure (NSP), i.e., "pushover analysis" (FEMA 273, 3.3.3). If the LDP indicates significant inelastic demand in a tower shaft, that tower will be evaluated using the NSP. This inelastic demand will be quantified in terms of demand-capacity ratios (DCRs), from the LDP (FEMA 273, 2.9.1.1). If any DCRs exceed 2.0, except for cab elements, the tower may be evaluated using an NSP. DCRs in excess of 2.0 were expected in the tower cabs, particularly at the window mullions. Significant inelastic response of the cab should not have much influence on the performance of the tower shaft. This is because of the small weight of the cabs relative to that of the shafts. Though the cabs provide a very critical function, they are similar to a penthouse appendage. More detailed evaluation of the cabs was performed if several cab members or connections had high DCRs.

Tower Foundation Assumptions

The tower foundations were modeled as fixed bases (FEMA 273, 3.2.2.6; FEMA 302, 5.4.4). Rotation at the base of the structure would cause small increases in moments due to P-delta effects. However, these structures are relatively lightweight and P-delta increases in loads should be negligible, so this effect was neglected. Neglecting base rotations also decreases calculated total deflections. In this evaluation, relative (not absolute) displacements were most critical for evaluating tower vulnerability.

Type L Tower Shaft Configuration and Model

The Salinas tower shaft is a 50-ft tall reinforced concrete frame and is square in plan. The tower shaft consists of four tapered bents (one at each corner) joined at the center of the top of the shaft. These column bents protrude out diagonally from the shaft corners, and taper from a maximum width of 82 in. at ground

level to a minimum width of 36 in. at the top of the shaft (Drawing S1; Corlett & Spackman 1966). The vertical portion of three bents can be seen in Figure 1. The horizontal portion of each concrete bent is 24-in. deep. The bents are 18-in. wide in both the vertical column and horizontal beam portions. The reinforced concrete floor slabs are supported by steel beams, which in turn frame into the concrete bents. These beams are constructed with shear studs so that they will behave as a composite section with the floor slabs. The floor slabs are 5-in. thick, consist of one-way construction, with two layers of number 4 bars at 12 in. on center, and span approximately 7 ft to the beams. Shear transfer from slab to beam is accomplished using ¾-in. welded studs at 12 in. on center along the beam centerline. Beams are typically 16B26 and 16WF40 (similar to currently available W16x26 and W16x40) and they span 21 ft to the four corner columns. The column bents are supported by reinforced concrete grade beams, which act both as spread footings and tie elements. Additionally, tie elements are provided along the diagonal of the structure, at the foundation, to prevent the columns from spreading apart under gravity loads.

The tower lateral-load-resisting system consists of concrete floor diaphragms that bear upon reinforced concrete moment frames. The frames are formed by the four corner column bents, which create two individual frames, one in each orthogonal direction at the diagonal of the shaft plan. These columns extend to the top of the tower shaft. The column bents are reinforced with three number 14 bars at each face (strong axis of columns) up to an elevation of 24 ft 6 in., plus three additional number 14 bars on each face up to an elevation of 12 ft 6 in. Four number 5 bars are also located on each face (weak axis of columns) that fan out at equal spacing over the entire elevation of the columns. Three number 11 bars at each face (strong axis) begin below the 24 ft 6 in. elevation and are spliced to the number 14 bars. Column shear reinforcement consists of number 4 bar ties at 9 in. on center for the bottom 25 ft and number 4 bars at 12 in. on center for the remaining column elevation. The horizontal beam portion of the bents are reinforced with three number 10 bars at the top and bottom of the beam. Additional reinforcement is provided at the corner of the bent. Shear reinforcement of the horizontal beam portion of the bent consists of number 3 bar ties at 12 in. on center.

Each floor level is connected at an intermediate point of the frame columns (bents). Lateral loads are transferred from the floor diaphragms to the columns by either compression bearing or by tension in embedded anchor bolts. The two moment frames cross each other at the center of the shaft roof in plan. A hinge is installed in each moment frame at this location that decouples the frames in the two orthogonal directions, allowing the frames to act independently. If the

hinge were not present, large torsional forces would be applied to the frame elements during lateral seismic motions.

The reinforced concrete bents were modeled using beam elements. The full gross cross-sectional area of the concrete was used in these elements, as this will produce the largest forces and moments. The pinned bent connection at the top of the shaft was modeled as a pinned connection in all three directions. The concrete floors and wide flange beams at each floor level are modeled as rigid diaphragms. The concrete floor slabs are designed to behave composite with the wide flange beams and the section properties used to model these elements use the effective width of the concrete slab.* The connections of the floor wide-flange beams to the bents are modeled as pinned connections in both principal axes of the beams. The shaft roof does not have a concrete slab, so this is not modeled as a rigid diaphragm. Figure 4 shows the finite element mesh that represents the shaft and cab of the 50 foot tall Salinas ATCT.

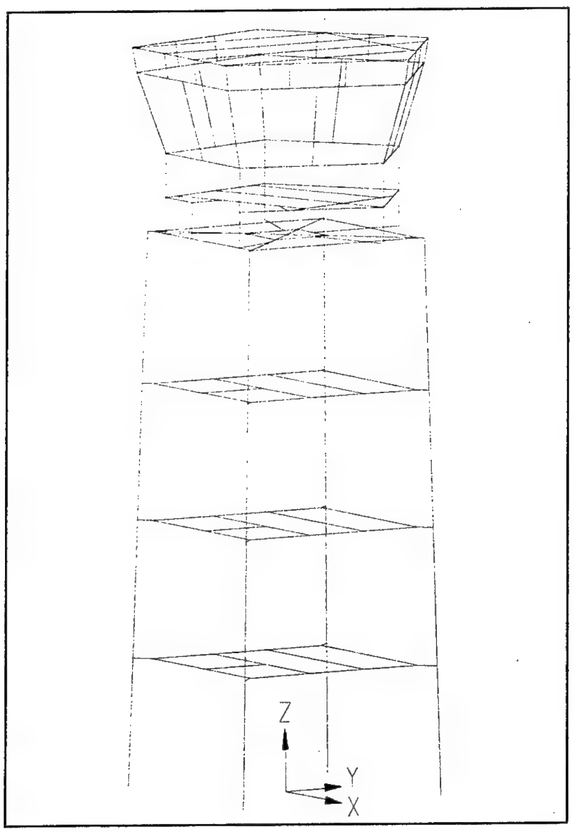

Figure 4. Finite element mesh for the 50-ft tall Salinas tower.

* AISC LRFD, I3.1.

Gravity and seismic loads are applied to the model by including the self-weight in appropriate structural elements and adding the effects of seismic loads through the use of a response spectrum. All self-weight beyond that in the structural members is added to the model by adding to the unit weight of beam members of the structure. Normally 50 percent of this self-weight was added to the beams at the perimeter of the structure and the other 50 percent to interior beams. The weight of wall cladding and stairs was distributed equally between the floor below and above the floor. The weight of interior partition walls and doors was distributed with one-third to the floor above and two-thirds to the floor below. Appendix A provides information on the calculation of weights distributed to each member. The distributed loads applied at each floor level, not including the self-weight of the members, are summarized in Table 2 for the Salinas tower.

The effects of horizontal torsion were considered by offsetting the center of mass of both self-weight and live loads 5 percent of the floor width in the direction perpendicular to the primary (100 percent response spectrum) lateral loading. The direction of mass offset is 180 degrees from the maximum lateral stiffness, so that the maximum distance, perpendicular to the primary load, is created between the center of stiffness and of mass (FEMA 273, 3.2.2.2). This offset of 5 percent was obtained by factoring the member weight and mass as shown in Appendix A. These factors apply only to the distributed floor weight, not the member self-weight or cladding weight. The gravity loads are applied in this manner for all floor levels. The actual loads placed on each member are summarized in Appendix A.

Table 2. Salinas Type L Tower Distributed Loads.

Floor Level	Partitions and Doors (lb)	Stairs (lb)	Floor System (lb)	Mech, Elec and Misc (psf)	Total at Floor (psf)	Exterior Cladding etc (lb/ft)
2nd	2662	2828	22171	15	93	195
3rd	2515	2828	22171	15	93	194
4th	3447	2761	23325	15	95	41
Top of Shaft	1304	2096	14824	5	43	94
Cab Floor	329	133	17796	5	51	33
Cab Roof	-	-	15071	5	25	13

Type L Tower Cab Configuration and Model

The tower cab is configured as a pentagon in plan. The lateral load resisting system of the tower cab consists of light structural steel moment frames. The cab columns are built-up structural tubes, which are 7-in. deep and 4-in. wide. The columns also serve as the window corner mullions. These will likely be the most

vulnerable members in the Type L towers. The cab floor is a steel deck with a cast-in-place concrete slab, supported with wide flange beams. The cab framing is connected to the shaft roof framing, which in turn is connected to the concrete frames by embedded anchor bolts. The cab roof is a steel deck supported by steel beams, which frame into the supporting cab columns (window mullions). The top and bottom of the windows are supported with channel sections that are oriented with their strong axis perpendicular to the plane of the window. At the center of each face of the cab, the windows are divided and supported by smaller interior mullions (S3 x 7.5).

The tower cab was modeled using beam elements for the mullions and beams. Mullion/beam connections are fixed at the base of the mullions only. Rigid diaphragms connect all members at the cab floor and roof levels, preventing relative horizontal deflection, while allowing vertical flexure. The cab roof and floor are both modeled as rigid diaphragms, even though only the cab floor has a concrete slab. The rigid diaphragm at the roof is slightly unconservative for members in the roof, because it will eliminate weak axis bending and axial loading on the beam elements at this level. However, these loads would be insignificant, plus the steel roof deck will in reality provide a degree of rigidity, especially in the weak axis direction of the beams, which is the direction of roof deck corrugation. The rigid roof diaphragm will more realistically provide the model performance that will more heavily load critical cab members.

Cab windows have often failed in past earthquakes because stress concentrations form where the brittle glass comes in contact with the frame. However, if the glass does not fail, and it effectively works with the window mullions and other window frame members, it will perform as a stiff shear wall. This condition will load the cab members below the window more severely, plus it will load the shaft more heavily. The stiffness of the cab will increase significantly, so that the natural period decreases, which will couple the first mode of the cab with the first mode of the shaft. This decrease in period will also increase the effective acceleration of the cab due to the shape of the response spectra. Therefore, the worst case shaft evaluation will be the case where all the cab windows act as fully effective shear walls. This is modeled by increasing the moment of inertia of the members around the perimeter of the window to equal the shear stiffness of the windows.

The cab of the 30-ft tall San Carlos tower is identical to the Salinas tower's cab, but will be more severely loaded at the cab because of the shorter tower height making it stiffer with a smaller natural period and higher effective acceleration. The cab will also be more severely loaded due to the higher spectral acceleration values at San Carlos. In particular, the San Carlos spectrum is greater at higher

periods that correspond to the natural period of the cabs. Therefore, Type L cab evaluation will be based on the San Carlos tower. Figure 5 shows the finite element mesh that represents the shaft and cab of the 30-ft tall San Carlos ATCT. Figure 6 shows the model for the cab by itself. This figure shows the shape and orientation of each structural member. The distributed loads applied at each floor level, not including the self-weight of the members, are summarized in Table 3 for the San Carlos tower.

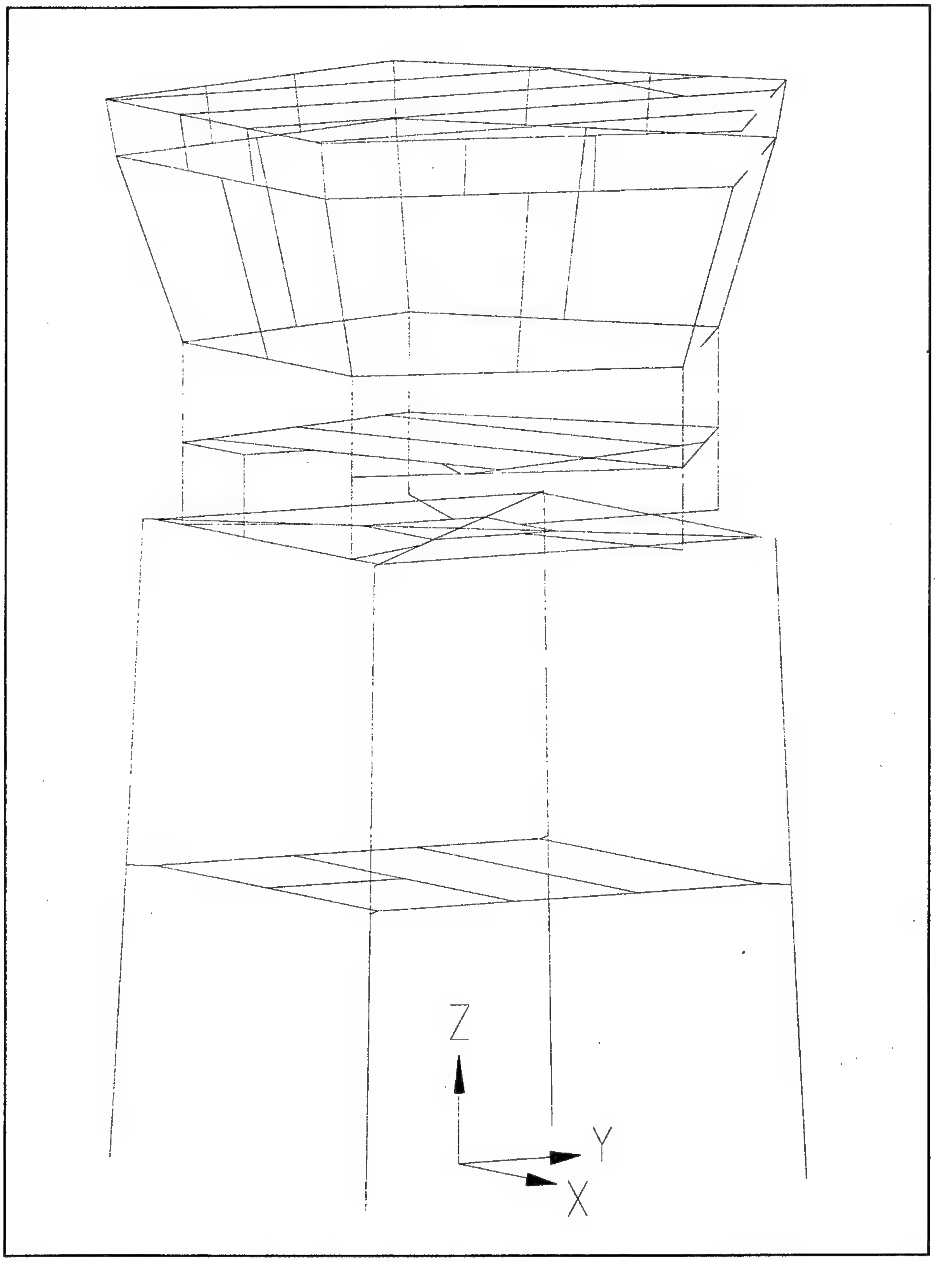

Figure 5. Finite element mesh for the 30-ft tall San Carlos tower.

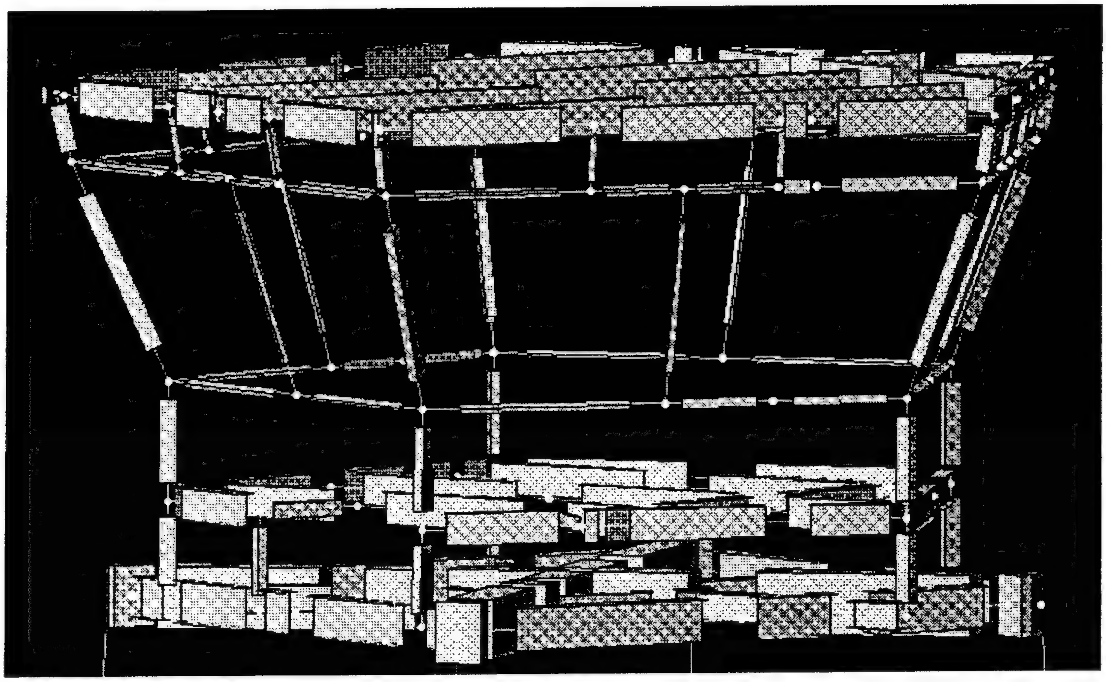

Figure 6. Finite element mesh for the cab of the San Carlos tower.

Table 3. San Carlos Type L Tower Distributed Loads.

Floor Level	Partitions and Doors (lb)	Stairs (lb)	Floor System (lb)	Mech, Elec and Misc (psf)	Total at Floor (psf)	Exterior Cladding etc (lb/ft)
2nd	3602	2761	23325	15	96	191
Top of Shaft	1304	2096	14824	5	46	94
Cab Floor	329	133	17796	5	51	33
Cab Roof	-	-	15071	5	25	13

Loads were distributed along the horizontal beam members at the cab floor, channels above and below the windows, mullions, and roof. The cab gravity loads are calculated as shown in Appendix A, and summarized for the cab floor and roof in Table 3. The seismic loads are applied using the response spectra as in the tower structure. As with the shaft, the weights and masses are distributed so as to provide 5 percent eccentricity of the distributed weight at the cab floor and roof levels. The weight and mass of exterior cladding and window glass are assumed to be uniformly distributed around the building perimeter without any offset (eccentricity). The cab window loads can be summarized as follows:

- Plate glass density = 161 pcf*

* 1994 AISC LRFD, Table 7-4, Weights and Specific Gravities, p 7-6.

- Single 3/8 in.-thick pane
- 161 pcf x 0.375 in./12 in./ft = 5.03 psf
- Total weight per window pane = 5.03 psf x 56.23 sf = 283 lb
- Distribute this weight equally to the members around the window perimeter, 283 lb/398 in. = 0.711 lb/in.

San Luis Obispo Tower Shaft Configuration and Model

The San Luis Obispo tower was built in 1988. The tower is 45 ft tall at the cab base, 48 ft at the cab floor and 64 ft at the top of the parapet. The shaft is square in plan with its sides measuring 20 ft in length. The tower construction consists of structural steel framing covered with insulated metal panel siding.

The vertical load resisting system consists of concrete-topped metal decking supported by wide flange steel beams and columns that bear on concrete foundations. Typical floor framing consists of 1-1/2-in. deep, 20 gauge metal decking, topped with 2-1/2-in. of concrete fill. The floor slab spans as much as 7 ft to the beams. Interior beams are W12 x 30, and the supporting beams around the perimeter that frame into the columns are W14 x 34, W16 x 40, and W16 x 45. The floor supporting steel beams do not include shear studs, so that the concrete is not designed to act as a composite section with the beams. The columns are W8 x 31 for the lower 23 ft of the tower shaft and W8 x 24 to the top of the shaft. The foundation consists of 6 ft x 6 ft x 18-in. deep spread footings under each column.

The tower lateral load resisting system consists of an eccentric braced steel frame, located symmetrically around the tower perimeter. These braces are L6 x 6 x ½ single angles with bolted connections. These braces frame into the beams so that the center 4-ft 3-in. portion of the beam is loaded eccentrically. These portions of the beam (a shear link) will carry large shear and moment loads and is intended dissipate energy in an earthquake. Additional W8 x 21 stub columns are added to the story below the shaft roof, below the base of the cab mullions. These work with the braces and beam below to form a partial truss for carrying gravity and seismic loads from the cab.

All structural members were modeled using beam/column frame elements. The concrete floors at each floor were modeled as rigid diaphragms. The shaft roof, however, does not have a concrete slab or even a roof deck, because the cab and

walkway around the cab perimeter cover the entire shaft. All connections between the columns, beams, and braces in the shaft were modeled as pinned. Figure 7 shows the finite element mesh for both the shaft and cab of the San Luis Obispo steel braced frame tower.

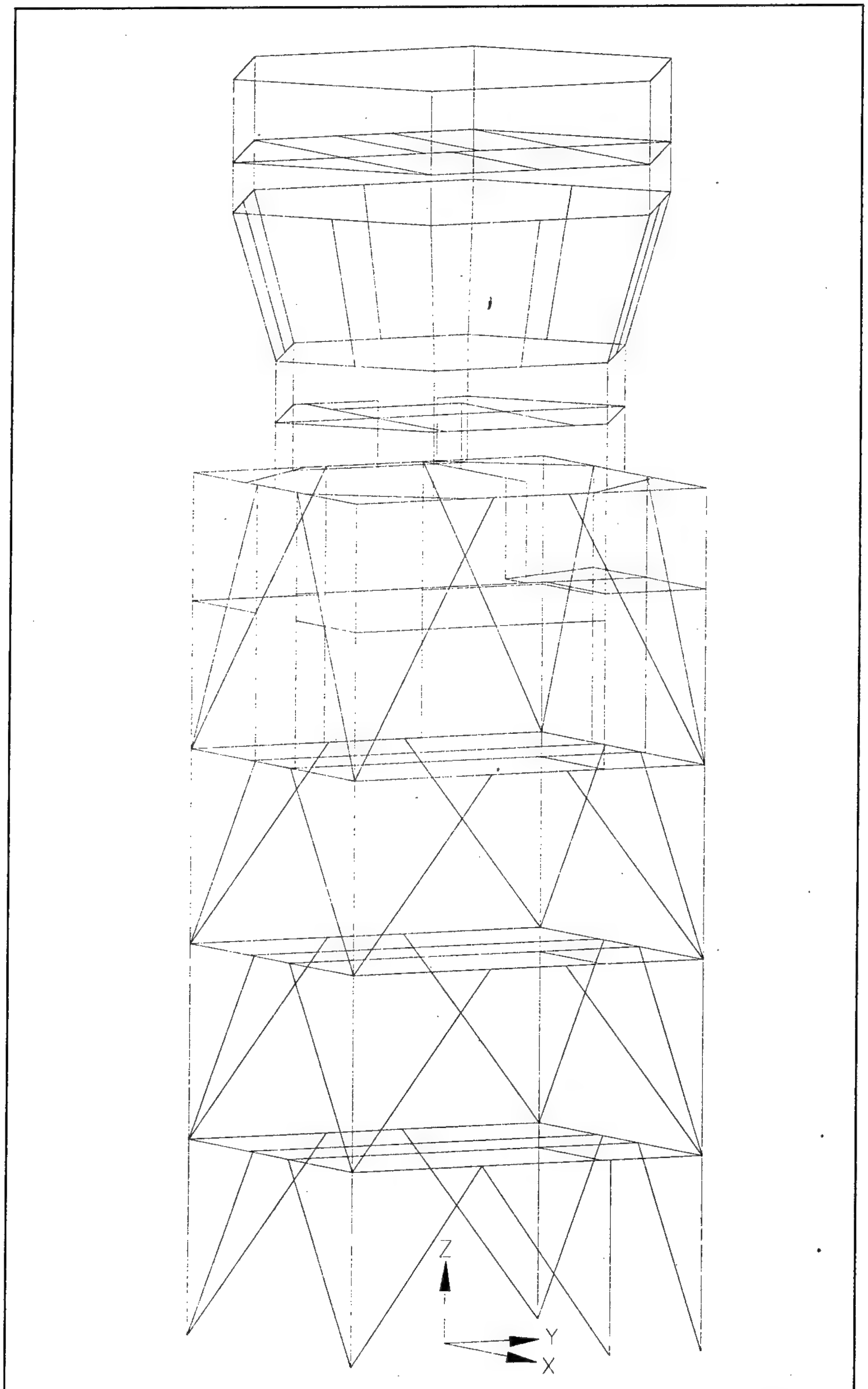

Figure 7. Finite element mesh for the 45-ft tall San Luis Obispo tower.

Gravity and seismic loads were applied to the model by including the self-weight in appropriate structural elements and adding the effects of seismic loads through the use of a response spectrum. All self-weight beyond that in the structural members was added to the model by adding to the unit weight of beam members of the structure. Normally 50 percent of this self-weight was added to the beams at the perimeter of the structure and the other 50 percent to interior beams. At the top of the shaft, 75 percent of the self-weight beyond the members was added to the perimeter beams and the remaining 25 percent to the interior beams. This level does not have a floor slab, so that most of the load at this level is near the building perimeter (from the walkway and cladding).

The weight of wall cladding, partition walls, and doors was distributed with one-third to the floor above and two-thirds to the floor below. The wall cladding distribution differs from the Type L distribution, which was distributed equally to the floor above and below. The weight of stairs was distributed equally between the floor above and floor below. Appendix C provides information on the calculation of weights distributed to each member. The distributed loads applied at each floor level, not including the self-weight of the members, are summarized in Table 4 for the San Luis Obispo tower.

As with the Type L towers, the effects of horizontal torsion were considered by offsetting the center of mass of both self-weight and live loads 5 percent of the floor width in the direction perpendicular to the primary lateral loading. The actual loads placed on each member to achieve this eccentricity are summarized in Appendix C.

Table 4. San Luis Obispo Distributed Loads*.

Floor Level	Partitions and Doors (lb)	Stairs (lb)	Floor System (lb)	Mech, Elec and Misc (psf)	Total at Floor (psf)	Exterior Cladding etc (lb/ft)
Intermediate 2	8200	4282	11941	15	77	36
Intermediate 3	7663	4282	11941	15	75	36
Junction	11621	4467	11690	15	89	47
Cab Access	2890	2541	3113	15	25	0
Top of Shaft	6657	1103	18809	5	75	27
Cab Floor	1062	193	10339	5	60	109
Cab Roof	0	0	5687	5	21	60

* The top of shaft floor system weight includes the walkway around the cab perimeter. The cab floor and cab roof cladding weights include the weight of the glass and the parapet cladding respectively. The areas of openings are included in areas used in calculating the "Total at Floor (psf)."

San Luis Obispo Tower Cab Configuration and Model

The San Luis Obispo tower cab is configured as a hexagon in plan. The lateral load resisting system of the tower cab consists of light structural steel moment frames. The cab columns are made from TS 8 x 4 x ½ structural tubing members. The columns also serve as the window corner mullions. These will likely be the most vulnerable members in the San Luis Obispo towers. The cab columns are welded to base plates, which are in turn welded to beams at the top of the shaft. The welded connection from this base plate to the beams is critical and details on the drawings are incomplete. Field inspection revealed that the base plate is welded around its entire perimeter to the supporting beam flange.* The cab floor is a steel deck with a cast-in-place concrete slab, supported with wide flange beams. These beams are connected to the cab columns with shear connections. The cab roof is a steel deck supported by wide flange beams, which frame into the supporting cab columns (window mullions), using shear connections. Horizontal structural tubing members (TS 7 x 7 x 3/16) span the cab columns above and below the windows and support the windows. These horizontal tubes are connected to the columns with full penetration grove welds all around the tube, making these moment connections. At the center of each face of the cab, the windows are divided and supported by smaller interior mullions made of TS 4 x 2 x 3/16 structural tubes. These interior mullions are welded† to supporting TS 7 x 7 x 3/16 members at both their tops and bottoms, making these moment connections. Above the cab roof, another horizontal TS 7 x 7 x 3/16 structural tube member frames into the cab columns at the top of the parapet wall. These members are also welded to the cab columns with a full penetration grove weld forming a moment connection.

The tower cab was modeled using beam/column frame elements for the columns and beams. Column/beam connections are fixed, at their base, bottom and top of windows and at the top of the cab parapet. Connections between the column and beams at the cab floor and roof levels are modeled as pinned because they are shear connections. Rigid diaphragms connect all members at the cab floor and roof levels, preventing relative horizontal deflection, while allowing vertical flexure. The cab roof and floor are both modeled as rigid diaphragms, even though only the cab floor has a concrete slab. The rigid diaphragm at the roof is slightly unconservative for members in the roof, because it will eliminate weak axis

* Inspected by Gary Benson (FAA representative at San Luis Obispo) on 11 August 1998.

† Based on field inspection.

bending and axial loading on the beam elements at this level. However, these loads would be insignificant, plus the steel roof deck will in reality provide a degree of rigidity, especially in the weak axis direction of the beams, which is the direction of roof deck corrugation. The rigid roof diaphragm will more realistically provide the model performance that will more heavily load critical cab members.

As explained in the Type L tower cab model assumptions, the worst case shaft evaluation will be the case where all the cab windows act as fully effective shear walls. This is modeled by increasing the moment of inertia of the members around the perimeter of the window to equal the shear stiffness of the windows. Figure 8 shows the San Luis Obispo cab model, including members at the shaft roof level and above. This figure shows the shape and orientation of each structural member.

Loads were distributed along the horizontal beam members at the cab floor, tubing above and below the windows, and roof and tubing at the top of the parapet. The cab gravity loads are calculated as shown in Appendix C and summarized for the cab floor and roof in Table 4. The seismic loads are applied using the response spectra as in the tower structure. As with the shaft, the weights and masses are distributed to provide 5 percent eccentricity of the distributed weight at the cab floor and roof levels. The weight and mass of exterior cladding and window glass are assumed to be uniformly distributed around the building perimeter without any eccentricity. The cab window loads can be summarized as follows*:

- Glass weight (1-in. thick pane) =15 psf† (180 pcf)
- Total weight per window pane = 15 psf x 5.4 ft x 7.8 ft = 634 lb
- Distribute one-third and two-thirds of this weight to the TS 7 x 7 x 3/16 tubes above and below the windows, respectively.

* Window weight calculations differed slightly from the Type L towers as shown here.

† 1994 AISC LRFD, Table 7-5, Weights of Building Materials, p 7-7.

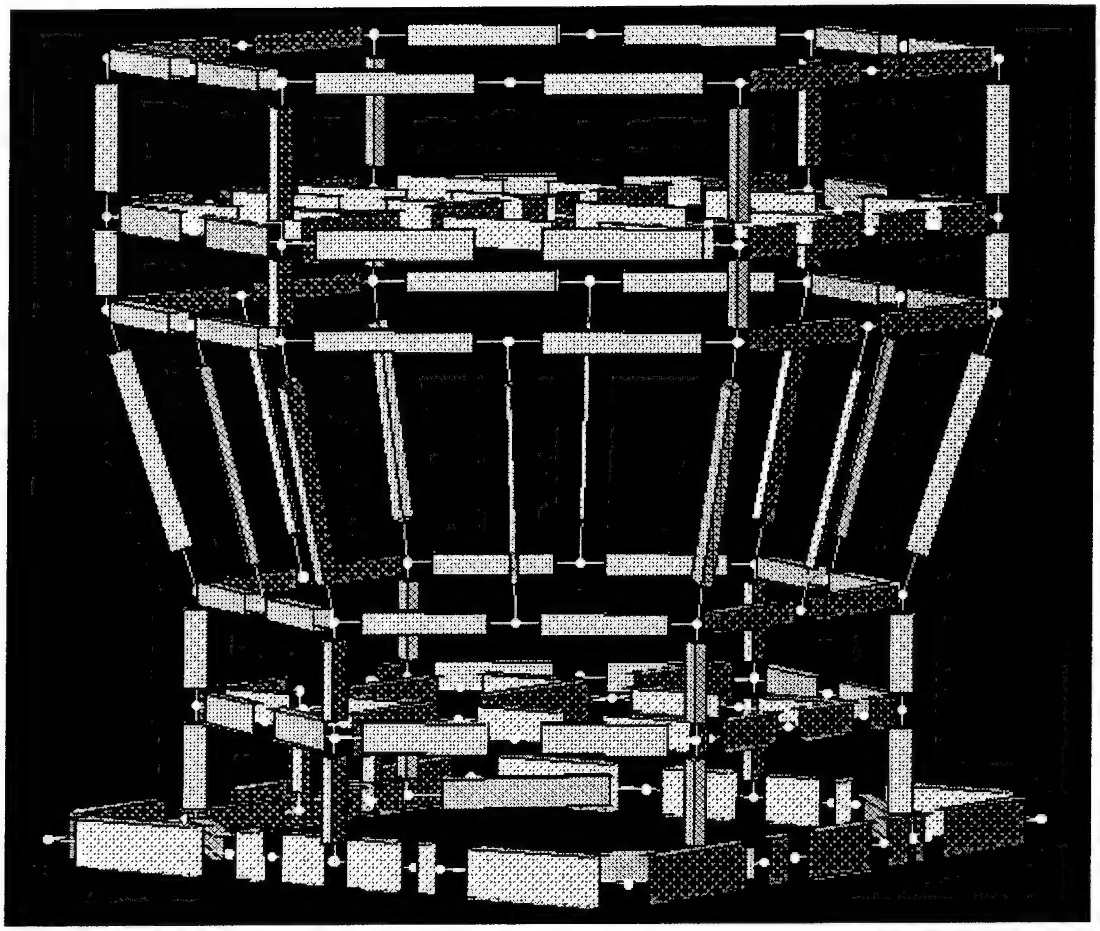

Figure 8. Finite element mesh for the cab of the San Luis Obispo tower.

Response Spectra Development

Evaluation response spectra were developed based on both the 1991 NEHRP (FEMA 222) guidance and the current 1997 NEHRP (FEMA 302). FEMA 222 is used because the results of these evaluations should be consistent with the earlier Standard ATCT evaluation in UBC Seismic Zones 3 and 4. FEMA 302 is used because it is current guidance with an improved more refined definition of seismic hazard. These two spectra are compared and are found to produce seismic hazard definitions that are reasonably consistent with one another. Finally, the FEMA 302 spectrum definition of seismic hazard is used for all evaluations.

1991 NEHRP (FEMA 222) Based Response Spectra

The modal seismic design coefficient, C_{sm} is given by Equation 1 (from FEMA 222, Equation 5-3):

$$C_{sm} = \frac{1.2 A_v S}{R T_m^{2/3}} \qquad \text{[Eq 1]}$$

where:

A_v = the effective peak velocity-related acceleration in g's (from FEMA 222, Map 2), and a value of 0.4 g was used for each tower.

S = the site coefficient (from FEMA 222, Table 3.1). An S2 soil profile was used in earlier evaluations with a value of 1.2, and is again used here.

R = the response modification factor (defined in FEMA 222, Table 3.3). Because this evaluation is based on a pushover analysis procedure, it was decided to evaluate them assuming elastic response by assuming an R factor of 1.0.

T_m = the modal period of vibration in seconds for the mth mode of the building.

The modal seismic design coefficient, C_{sm}, is limited to a maximum value, C_{smMax}, given by Equation 2:

$$C_{smMax} = \frac{2.5 A_a}{R} \qquad \text{[Eq 2]}$$

where:

A_a = the effective peak acceleration in g's (from FEMA 222, Maps 1); a value of 0.4 g was used for each tower.

The FEMA 222-based evaluation spectra are plots of these modal seismic design coefficients, C_{sm}, shown in Figure 9.

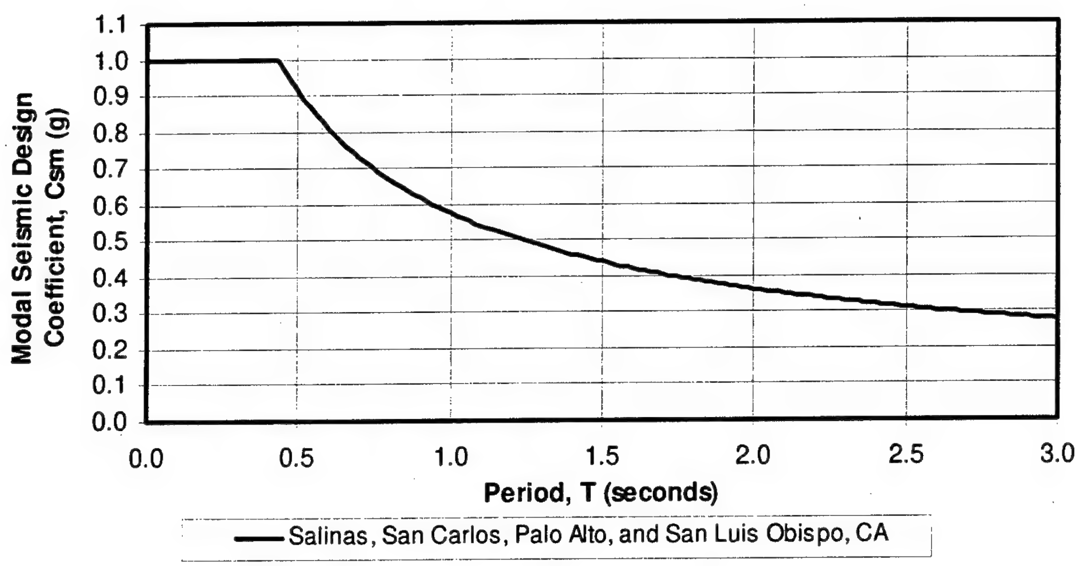

Figure 9. 1991 NEHRP-based evaluation response spectra, C_{sm}

1997 NEHRP (FEMA 302)-Based Response Spectra

The Seismic Performance Objective for all of these towers is Substantial Life Safety,* because all of them are noncritical power facilities. This performance objective equates to the Seismic Use Group I in FEMA 302. Because soil conditions are unknown, a Site Class D is conservatively assumed (FEMA 302, 4.1.2.1). The maximum considered earthquake design spectrum is defined for each tower based on the variables given in Table 5. These are based on the FEMA 302 design maps for short period, S_S, and at 1 second, S_1, for 5 percent damped spectral response acceleration (FEMA 302, 4.1.1). The spectral quantities used to define these spectra are given in Table 5 for each tower. For Site Class D values of site coefficients, F_a and F_v are 1.0 and 1.5 respectively for all the towers being evaluated here (FEMA 302, Tables 4.1.2.4a and 4.1.2.4b).

* U.S. Army Corps of Engineers, Seismic Design for Buildings, TI 809-04, 1998.

Table 5. FAA response spectra calculations based on FEMA 302.

California Location	S_S (g)	S_1 (g)	S_{MS} (g)	S_{M1} (g)	S_{DS} (g)	S_{D1} (g)	T_0 (sec)	T_S (sec)
Salinas	1.50	0.60	1.50	0.90	1.00	0.60	0.12	0.60
San Carlos	1.60	0.90	1.60	1.35	1.07	0.90	0.17	0.84
Palo Alto	1.50	0.60	1.50	0.90	1.00	0.60	0.12	0.60
San Luis Obispo	1.50	0.60	1.50	0.90	1.00	0.60	0.12	0.60

The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1 second (S_{M1}), adjusted for site class effects are calculated as follows (FEMA 302, Equations 4.1.2.4-1 and 4.1.2.4-2):

$$S_{MS} = F_a S_S \qquad \text{[Eq 3]}$$

and

$$S_{M1} = F_v S_1 \qquad \text{[Eq 4]}$$

These values define the elastic spectra. The values are reduced to define design earthquake spectral response acceleration at short periods, S_{DS}, and at 1-second period, S_{D1} as follows (FEMA 302, Equations 4.1.2.5-1 and 4.1.2.5-2):

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \text{[Eq 5]}$$

and

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \text{[Eq 6]}$$

From these terms, design response spectra are developed for each of the tower locations. For the natural period of the structure (T), this spectrum defines values of effective acceleration. The three regions of this spectrum are defined as follows:

- For periods less than or equal to T_0, the design spectral acceleration, S_a, shall be (FEMA 302, Equation 4.1.2.6-1):

$$S_a = 0.6 \frac{S_{DS}}{T_0} T + 0.4 S_{DS} \qquad \text{[Eq 7]}$$

- For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.

- For periods greater than T_S, the design spectral response acceleration, S_a, shall be (FEMA 302, Equation 4.1.2.6-3):

$$S_a = \frac{S_{D1}}{T} \qquad \text{[Eq 8]}$$

where

T = the fundamental period of the structure in seconds

T_0 = $0.2S_{D1}/S_{DS}$

T_S = S_{D1}/S_{DS}.

Figure 10 shows these design response spectra for the towers in Table 1. Reliability factors, ρ, were calculated, and found to be less than 1.0 for each tower, so that a value of 1.0 is used (FEMA 302, 5.2.4.2). This is a factor for the extent of structural redundancy.

The modal seismic response spectra, C_{sm}, are calculated from the design response spectra (Figure 10), using the following equation (FEMA 302, Equation 5.4.5-3):

$$C_{sm} = \frac{S_a}{R/I} \qquad \text{[Eq 9]}$$

In these evaluations, R and I values of 1.0 are used so that the modal seismic response coefficient, C_{sm}, is the same as the design response spectra, S_a. However, R values of 1 are used for each tower because the response spectra generated is used for pushover analysis where the analysis assumes a linear response. Member and connections being evaluated may have demand-capacity ratios, DCRs as great as 2.0, and be considered to pass evaluation without further evaluation. For those members with DCRs greater than 2.0, hinges will be placed at joints, and further load applied.

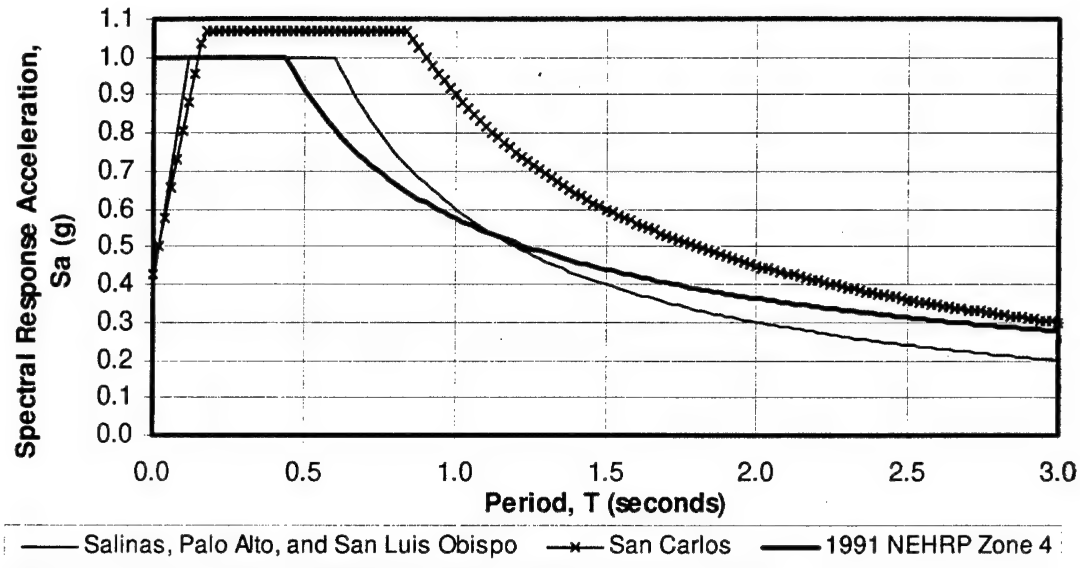

Figure 10. Maximum considered earthquake design response spectra.

Figure 10 shows the FEMA 222 spectrum plotted along with the FEMA 302 spectra for all the towers. This figure demonstrates that the new spectra are reasonably consistent with the spectrum used in the earlier evaluations. The San Carlos spectrum is an exception, where the spectral acceleration values are significantly greater at periods greater than 0.5 seconds. The primary modes that dominate the response of the cab are at periods of 1.27 and 0.78 seconds. At these periods, the spectral accelerations are 0.71 and 1.07 g for the FEMA 302 San Carlos spectrum, 0.47 and 0.77 g for the other FEMA 302 location spectra, and 0.49 and 0.68 g for the FEMA 222 spectrum. Modes of vibration that control response of the shaft are only slightly influenced by the difference in spectra because their periods (0.60 and 0.55 seconds for Salinas and 0.39 and 0.38 seconds for San Luis Obispo) fall within the period range where the spectral accelerations are 1.0 g (slightly less for FEMA 222). The San Carlos tower was evaluated with the spectrum shown in Figure 10, with a slightly greater spectral amplitude of 1.07 g for the periods that control the response of the shaft (0.45 and 0.28 seconds). However, for the Type L towers, the taller Salinas tower has a more critical shaft, so the higher amplitude San Carlos spectrum does not control the vulnerability of the Type L shaft components. Therefore, the vulnerability of the shaft components based on the FEMA 302 spectra is consistent with their vulnerability had they been based on FEMA 222 spectrum. In a similar manner, the difference in vulnerability of particular structural components, between the 1991 and 1997 NEHRP spectra, can be evaluated. For any member or connection, the mode(s) of vibration that dominate their vulnerability can be determined (based on mode shapes and participating mass), and the spectral acceleration between

the 1991 and 1997 spectra compared to assess that component's difference in vulnerability.

Load Combinations

Combination of Load effects were evaluated for the worst-case combinations of horizontal and vertical loads, together with gravity loads. The effect of seismic loads, E, when the horizontal and gravity loads are additive is represented by (FEMA 302, Equation 5.2.7-1):

$$E = \rho_x Q_E + 0.2S_{DS}D \qquad \text{[Eq 10]}$$

where

E = the effect of horizontal and vertical earthquake-induced forces

D = the effect of dead load

Q_E = the effect of horizontal seismic forces.

For all the control towers, at all floor levels $\rho_x = 1$, so that

$$E = Q_E + 0.2S_{DS}D \qquad \text{[Eq 11]}$$

The effect of seismic loads, E, when the horizontal and gravity loads counteract each other was represented by (FEMA 302, Equation 5.2.7-2):

$$E = \rho_x Q_E - 0.2S_{DS}D = Q_E - 0.2S_{DS}D \qquad \text{[Eq 12]}$$

The gravity loads (Q_G) were combined as follows when the effects of gravity are additive with seismic loading (FEMA 273, 3.2.8, Equation 3.2):

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad \text{[Eq 13]}$$

When the effects of gravity counteract seismic loads, Q_G becomes (FEMA 273, Equation 3.3):

$$Q_G = 0.9Q_D \qquad \text{[Eq 14]}$$

where:

Q_D = the full weight of the structure

Q_L = the effective live load equal to 25 percent of the unreduced design load plus 100 percent of partition walls and attached equipment

Q_S = the effective snow load, which is taken as 20 percent (25 percent in 1997 UBC, Section 1612.3.2) of the design snow load when the design snow load is at least 30 psf. For all tower locations shown in Table 1, the design snow load is less than 30 psf, so the effective snow load was zero for all towers being evaluated here.

The effects of gravity load (dead, live, and snow) and seismic forces were combined as follows when the effect of gravity and seismic loads are additive by combining Equations 11 and 13:

$$Q_G + E = 1.1(Q_D + Q_L + Q_S) + Q_E + 0.2S_{DS}D \qquad \text{[Eq 15]}$$

The weight calculations summarized in Tables 2–4 include the actual dead load, Q_D, and live load, Q_L. These combined loads, Gravity, are the gravity loads, D, that were multiplied by $0.2S_{DS}$ to calculate the vertical seismic loads. The effective snow load, Q_S, is zero for all the tower locations in Table 1. Using the values of S_{DS} in Table 5, Equation 15 becomes the following for all the towers in Table 1, except for San Carlos:

$$Q_G + E = 1.1(Gravity) + Q_E + 0.2S_{DS}(Gravity) = 1.3(Gravity) + Q_E \qquad \text{[Eq 16]}$$

The San Carlos tower has a spectrum response acceleration at short periods, S_{DS} of 1.07 g, so that Equation 15 becomes the following:

$$Q_G + E = 1.1(Gravity) + Q_E + 0.2S_{DS}(Gravity) = 1.314(Gravity) + Q_E \qquad \text{[Eq 17]}$$

The effects of gravity load (dead, live, and snow) and seismic forces were combined as follows when the effects of gravity counteract seismic loads, by combining Equations 12 and 14:

$$Q_G - E = 0.9(Q_D + Q_L) - Q_E - 0.2S_{DS}D \qquad \text{[Eq 18]}$$

The weight calculations summarized in Tables 2 through 4 include the actual dead load, Q_D, and live load, Q_L. Equation 18 includes the actual live loads, so that the combined loads were the same Gravity value as in Equations 16 and 17. Vertical seismic load is the same as in Equation 16, but with the opposite sign because seismic forces counteract the effects of gravity. Equation 18 became the following for all the towers in Table 1, except for San Carlos:

$$Q_G - E = 0.9(Gravity) - Q_E - 0.2S_{DS}(Gravity) = 0.7(Gravity) - Q_E \qquad \text{[Eq 19]}$$

The San Carlos tower has spectral response acceleration at short periods, S_{DS} of 1.07 g, so that Equation 18 became the following:

$$Q_G - E = 0.9(Gravity) - Q_E - 0.2S_{DS}(Gravity) = 0.686(Gravity) - Q_E \qquad \text{[Eq 20]}$$

The member and connection forces and deflections were calculated by combining the square root of the sum of the squares (SRSS) of the various modal contributions of the response spectrum analysis of each tower (FEMA 302, 5.4.8). The finite element software used for this analysis, SAP2000, has an option to combine the modal contributions based on the SRSS, and this feature was used here.

A response spectrum analysis was performed with a model that includes enough modes in both orthogonal (horizontal) directions so that at least 90 percent of the building mass was included in the participating mass (FEMA 302, 5.4.3). The modal contributions to calculating the member and connection forces, base reactions, and displacements were combined by the SRSS. Each tower was evaluated by applying 100 percent of the lateral response spectrum based loading in one direction and 30 percent of the spectrum in the perpendicular lateral direction (FEMA 273 3.2.7).

Story Drift and P-Delta Effects

Story drifts were calculated directly from this analysis. Story drifts are the maximum difference in displacements at one floor level, δ_x, and the floor level below, δ_{x-1}, for a given column. For the tower cab, this became the maximum difference

in displacement at the cab roof and floor for a given window corner mullion. These displacements are the SRSS of the calculated displacements in the x and y direction (orthogonal horizontal directions) in the model.* The x and y displacements were calculated by SAP2000 based on the SRSS of each of the modal contributions of displacements at the particular joint in question. In this elastic analysis (no pushover), the elastic deflections should be amplified as follows (FEMA 302, Equation 5.3.7.1):

$$\delta_x = \frac{C_d \delta_{xe}}{I} \quad \text{[Eq 21]}$$

where:

C_d = the deflection amplification factor—a value of 4.0 could be used for all towers (FEMA 302, Table 5.2.2)

δ_{xe} = the deflection determined by elastic analysis

I = the occupancy importance factor, which for Seismic Use Group I is 1.0.

The amplification expressed in Equation 21 applies only to the cab steel frame elements, as the concrete bent elements in the Type L towers remain essentially elastic, even based on an analysis with an R value of 1. However, had the analysis and resulting deflections been based on seismic forces defined by the R values given in FEMA 302, Table 5.2.2 (R value of at least 4), these deflections would be proportionately smaller. Therefore, the deflections used to evaluate drift limitations were modified as follows, where the effects of R value and deflection amplification factor, C_d, canceled each other out, because calculated elastic deflections were based on R values of 1:

$$\delta_x = \frac{C_d \delta_{xe}}{RI} = \frac{(4)\delta_{xe}}{(4)(1)} \quad \text{[Eq 22]}$$

* Table 11 gives examples of such displacements for the Salinas tower.

These deflections, for a given column, were used to calculate story drifts, Δ_c, as follows:

$$\Delta_c = \delta_x - \delta_{x-1} \qquad \text{[Eq 23]}$$

These calculated story drifts must be less than the allowable story drifts, Δ_a, calculated as follows (FEMA 302, 5.2.8, Table 5.2.8 and 5.3.7.1):

$$\Delta_a = 0.020 h_{sx} \qquad \text{[Eq 24]}$$

where:

h_{sx} = the story height below level x.

P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient, θ, as determined by Equation 25, is equal to or less than 0.10 (FEMA 302, 5.3.7.2):

$$\theta = \frac{P_x \Delta_c}{V_x h_{sx} C_d} \qquad \text{[Eq 25]}$$

where:

P_x = the total weight above or at level x

V_x = the seismic shear force acting between level x and x – 1

C_d = the deflection amplification factor. A value of 1 is used because the story drifts, Δ_c, are based on deflections, δ_x, that have not been amplified with values of C_d as explained above.

For the towers evaluated here, the stability coefficient, θ, was well below 0.10 for the tower shafts and cabs. No correction to calculated story drifts, Δ_c, was needed for P-delta effects.

3 Analysis Steps and Cases

The following steps were taken to conduct an analysis of each controlling tower configuration or case using SAP2000 finite element software. Several cases were evaluated because different tower configurations or load directions were more critical for various members or connections than others, and this approach allowed a thorough evaluation of the Type L and San Luis Obispo tower vulnerabilities. Steps 1 through 3 were followed for each analysis case. The remaining steps would have been used for only the controlling cases if a push-over analysis were needed to evaluate the vulnerability of a tower. Such evaluations would be based on the analysis cases where a member or connection DCR both exceeds 2.0 and is the greatest for all cases of the tower being evaluated. Push-over analysis was not needed for any of the tower evaluations presented here, but the procedure is described because it shows the context for the analysis conducted. In the San Luis Obispo tower evaluation, many compression braces buckled. Step 5a was used in analysis case SLO3a to evaluate this tower by removing compression braces, so that the remaining braces were tension only.

Analysis Steps

The following steps outline the approach used to evaluate each controlling configuration of the Type L or San Luis Obispo towers. More than one step was taken in most SAP2000 analysis runs, and these steps are shown in terms of the Run number.

Run 1

1. Gravity load alone was first applied to the towers.

2. A modal analysis was conducted to define the periods and mode shapes. SAP2000 uses this information to define the distribution of lateral seismic loads applied to the structure. These loads are based on mode shapes and period of each mode and the effective acceleration based on the spectral acceleration for each mode defined by the response spectrum.

3. The model was analyzed for the horizontal earthquake ground motions using the response spectra analysis procedure. Vertical ground motions were accounted for by multiplying the gravity load effect by the factors defined in Equations 16, 17, 19, and 20.

4. When a member or connection has a DCR greater than 2.0, for the gravity (step 1) and full elastic earthquake spectra (step 3), a seismic factor (F_1) was calculated. This is the factor of seismic load that, together with gravity, gives a DCR equal to 1.0 for the failed component. The loads and deflections for the entire structure were recorded at this factored load. If, for example, all the corner mullions near their bases have a similar DCR and it is over 2.0, an F_1 value should be determined for which the average corner mullion DCR is equal to 1.0. This is the point at which mullion yielding occurred and a hinge has formed at the base of each mullion. This was done for the San Carlos Type L tower evaluation, where F_1 equaled 0.1055. However, this resulted in a collapse mechanism and push-over analysis was not needed.

Run 2

5. If needed the model would be modified to reflect the yielded condition and the analysis continued as summarized below:

    a. If compression members buckle, their capacity in compression would be reduced to zero. Under this condition, these members would be removed. Modal analysis would be repeated with the modified model. The mode shapes and frequencies would be checked against the initial modal analysis (Run 1), to validate behavior. The new structure would be reanalyzed for the total gravity and seismic loads as presented below. However, for the control towers with relatively little redundancy, it is unlikely that they would be able to pick up the full gravity and seismic load after a critical highly stressed member buckles.

    b. If members yield in tension or bending, the applied loads at these locations would not be allowed to increase further with increasing displacements (i.e., a flat stress versus strain curve for additional loading). This would be the case if, for example, the mullions fail in bending near their base. Hinges would be placed at these locations, and modal analysis would be repeated with the modified model. The mode shapes and periods would be checked against the initial modal analysis (Run 1), to validate behavior. The new structure would be analyzed for the additional seismic loads only (loads beyond the factored seismic loads, F_1). Member

loads from this analysis would be added to the factored load determined in Step 4, and new DCR values determined.

6. If needed, the seismic analysis would be repeated with the seismic loads only (both horizontal and vertical). If new yielding of members or connection failure occurs before the full seismic load is carried, a second seismic factor (F_2) is calculated for that point at which the second yielding occurs. The applied load would be factored up (F_3,...,F_n) until further yielding, collapse of the structure, or the full seismic loading is carried.

7. Steps 5 and 6 are repeated, as needed, until further yielding, collapse, or the full seismic load is carried.

8. The results of the various load steps were summed in a spreadsheet. The resulting applied loads for critical members and connections were compared against the capacity of that component in a MathCAD* model that represents the capacity of the component at its loaded condition.

9. Total lateral displacements were presented for the maximum displacement at each critical floor level or elevation in the tower.

10. If a push-over analysis was carried out, story shear would be plotted with respect to story drift (maximum horizontal displacement) at critical floors or elevations in the tower. Calculated drifts would be compared with allowable story drift for these structures (0.02 h_{sx}).

Analysis Cases

The model configurations and loading directions, θ_L (shown in Tables 6 and 7) were used to evaluate the Salinas and San Carlos Type L and San Luis Obispo braced frame towers.

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* MathCAD software is a product of MathSoft Inc, Cambridge, MA.

Table 6. Type L Analysis Cases.

Case #	Critical Component	Tower Location	Tower Height	Windows Acting as Shear Walls	Full Spectrum Direction, θ_L*
L1-50	Shaft	Salinas	50 ft	All	90 deg.
L2-50	Shaft	Salinas	50 ft	All	45 deg.
L3-50	Shaft	Salinas	50 ft	All	0 deg.
L4-50	Shaft	Salinas	50 ft	One	19.3 deg.
L5-30	Cab	San Carlos	30 ft	None	0 deg.
L6-30	Cab	San Carlos	30 ft	None	90 deg.
L7-30	Cab & Shaft	San Carlos	30 ft	One	19.3 deg.
L8-30	Cab & Shaft	San Carlos	30 ft	All	90 deg.
L9-30	Cab & Shaft	San Carlos	30 ft	All	45 deg.
L10-30	Cab & Shaft	San Carlos	30 ft	All	0 deg.
L11-50	Cab & Shaft	Salinas	50 ft	None	90 deg.
L12-30	Cab/upgrade	San Carlos	30 ft	None	90 deg.
L13-50	Shaft/upgrade	Salinas	50 ft	All	90 deg.

Table 7. San Luis Obispo Analysis Cases.

Case #	Critical Component	Tower Height	Windows Acting as Shear Walls	Full Spectrum Direction, θ_L*
SLO1	Shaft	45 ft	All	90 deg.
SLO2	Shaft	45 ft	All	45 deg.
SLO3	Shaft	45 ft	All	0 deg.
SLO3a	Tension braces only	45 ft	All	0 deg.
SLO4	Cab & Shaft	45 ft	One	16.7 deg.
SLO5	Cab	45 ft	None	90 deg.
SLO6	Cab	45 ft	None	45 deg.
SLO7	Cab	45 ft	None	0 deg.

* Angle of primary loading, counter clockwise to the X-axis.

4 Type L Analysis Results

All Type L towers were analyzed using SAP2000 and MathCAD member and connection evaluation files. The SAP2000 analysis modeled the towers and defined component demand in terms of displacements and member forces and moments. The MathCAD files determine member and connection capacity and demand capacity ratios, DCR, based on the SAP2000 forces and moments. The first section of these results focuses on the tower shaft evaluation, which is controlled by the taller 50-ft Salinas tower. The second section focuses on the cab evaluation, which is controlled by the shorter 30-ft San Carlos tower. Table 6 summarized all the cases used to evaluate the Type L towers. Each section in this report presents the modal analysis results, deflections, story drifts, forces, and moments of critical members and connections, and the resulting DCR. The towers may fail evaluation based on either exceeding drift limits, or high DCRs that lead to collapse of the towers. Code-based design resistance factors have been dropped (i.e., set to 1.0) for the purpose of evaluating structural members. The design resistance factors are included in the evaluation of connections, as such failure must be prevented because they would fail in a more brittle manner than the structural members.

Salinas Tower – Shaft Members and Connections

The most critical condition in the Salinas shaft evaluation is the extreme condition where the cab windows all remain intact and act as fully effective shear walls (analysis Cases L1 through L3 and L13). This will decrease the fundamental period of the structure and will increase the effective acceleration and therefore load the shaft members and connections more severely. The assumption of the windows acting as fully effective shear walls is unrealistic, but it provides an upper bound basis for evaluating the effect of the windows remaining intact. This effect is modeled by defining the shear stiffness of the window if it acts as a shear block. The properties of all members around the window perimeters are then increased so their bending stiffness equals the combined stiffness of the windows as shear blocks, plus the actual bending stiffness of the members. In the actual Type L tower cabs, the connections between the mullions and channels above and below the windows are pinned, but these are changed to “fixed” to represent the effect of the windows acting as shear walls. Analysis Cases L1, L2,

and L3 evaluate the performance of the Salinas tower with the windows acting as shear walls with the full seismic response spectrum acting at 90, 45, and 0 degrees to the X-axis (θ_L). The X-axis orientation is shown on the model plot in Figure 4. This axis is along the front face of the tower as shown on Drawing S2 (Corlett & Spackman 1966) for all three Type L towers.

Another possible case is the condition where only one window remains intact. Case L4 represents this condition where only one window acts as a shear wall. The location of the center of mass (for 5 percent accidental eccentricity) is placed farthest away from the plane of this shear wall window. This will create the greatest distance between the center of mass and center of stiffness in the plan of the building and will create the greatest torsional response of the tower shaft. The resultant direction of the 100 percent full seismic spectrum loading plus the 30 percent orthogonal spectrum is parallel to this shear wall window, to create the worst torsional response. The window mullion at the stairway (Col 5 on Drawing S3 and S4, section 9; Corlett & Spackman 1966) has a much deeper cross-section than the other mullions. This mullion is 12-in. deep at the base compared to the others, which are 7 in. Therefore, this mullion tends to attract more load. This shear wall mullion is placed along side the stiffer mullion to further increase the distance between the center of stiffness and center of mass in plan.

Finally, Case L11 is the 50-ft Salinas tower without any shear walls. This model was evaluated for the full seismic spectrum at only 90 degrees because this provided the most severe loading for critical shaft components.

Modal Analysis Results and Deflections

Table 8 presents the primary modes of vibration for the Type L, Cases L1, L2, and L3 evaluations. The cumulative participating mass shows that a much greater portion of the mass participates in the first X and Y lateral modes than in the L11 case (Table 9), for the same tower without the windows acting as shear walls. This demonstrates that the cab is more coupled with the tower shaft vibration in the first mode, and this will more heavily load the shaft components due to higher effective accelerations.

Table 10 summarizes maximum lateral deflections at each floor level and other key locations in the control towers for the shaft evaluation cases. All deflections are the SRSS of the total X and Y lateral deflections. The shaft deflections are greater for the shear wall window cases (L1 – L3) because of the coupling and higher effective accelerations described above. Cab deflections for the cases of

Table 8. Salinas (Type L – 50 ft) Shaft Evaluation Modal Analysis Results, L1, L2, and L3.

Mode #	L1, L2, L3 Period (sec)	Cumulative Participating Mass (%) X-dir	Y-dir	Z-dir	Mode of Vibration
1	0.724	5.2	0.7	0.0	1st Torsion
2	0.603	5.5	52.0	0.1	1st Y-Lateral
3	0.549	59.7	52.0	0.1	1st X-Lateral
4	0.353	63.4	59.1	0.3	2nd Torsion/Y-Lateral
5	0.341	65.7	67.4	0.8	2nd Y-Lateral/Torsion
6	0.211	67.9	67.4	0.8	2nd X-Lateral/Torsion
7	0.176	67.9	68.6	11.0	1st Vertical/cab floor & roof
8	0.166	68.1	68.6	11.1	3rd Torsion
9	0.134	68.8	69.1	11.1	2nd Vertical/cab roof & floor
10	0.124	74.7	69.5	11.3	3rd X-Lateral/cab rocking
11	0.119	74.7	75.1	12.0	3rd Y-Lateral/cab rocking
12	0.108	75.9	80.3	12.0	4th Y-Lateral/cab vertical
13	0.105	80.1	80.4	12.1	4th X-Lateral/cab vertical
17	0.097	83.6	81.6	20.3	3rd Vertical/4th floor & roof
20	0.094	83.6	82.5	31.3	4th Vertical/2nd & 3rd floor
31	0.077	85.2	85.6	39.6	4th Torsion
56	0.048	87.5	92.3	46.6	4th Y-Lateral
58	0.046	94.0	93.7	46.6	4th X-Lateral

Table 9. Salinas (Type L – 50 ft) Shaft Evaluation Modal Analysis Results, L4 and L11.

Mode #	L11 Period (sec)	Cumulative Participating Mass (%) X-dir	Y-dir	Z-dir	Mode of Vibration	L4 Period (sec)
1	1.351	4.6	3.5	0.0	1st Torsion	1.342
2	1.300	6.6	12.6	0.0	1st Y-Lateral	1.006
3	0.828	14.8	12.6	0.0	1st X-Lateral	
4	0.550	18.9	12.9	0.0	2nd Torsion/X-Lateral	0.653
5	0.468	21.5	64.2	0.1	2nd Y-Lateral	0.500
6	0.463	66.0	66.7	0.1	2nd X-Lateral	0.463
11	0.206	66.7	66.8	10.0	1st Vertical/cab floor & roof	0.204
12	0.189	67.3	66.9	10.0	3rd Torsion	
25	0.113	74.6	71.8	12.9	3rd X-Lateral/cab rocking	0.097
26	0.111	81.5	74.6	13.0	3rd X-Lateral/cab rocking	0.112
27	0.106	81.8	79.7	13.1	3rd Y-Lateral/cab rocking	0.108
32	0.097	83.2	81.2	21.5	2nd Vertical/4th floor	0.094
34	0.095	83.2	85.3	21.9	3rd Y-Lateral/cab vertical	
35	0.094	83.2	85.3	32.4	3rd Vertical/2nd & 3rd floor	0.094
39	0.091	83.3	85.5	38.1	4th Vertical/shaft roof	0.091
44	0.076	85.3	85.6	38.6	4th Torsion	0.077
69	0.048	87.5	92.1	46.6	4th Y-Lateral	0.048
71	0.046	94.0	93.7	46.6	4th X-Lateral	0.046

Table 10. Salinas Shaft Evaluation Selected Horizontal SRSS Deflections.

Location	L1 Joint #	L1 δ_{xe} (in.)	L2 Joint #	L2 δ_{xe} (in.)	L3 Joint #	L3 δ_{xe} (in.)	L4 Joint #	L4 δ_{xe} (in.)	L11 Joint #	L11 δ_{xe} (in.)
2nd Floor, δ_1	204	0.25	204	0.31	204	0.35	212	0.34	204	0.21
3rd Floor, δ_2	304	1.00	304	1.21	304	1.37	312	1.29	304	0.86
4th Floor, δ_3	404	2.15	404	2.52	404	2.83	412	2.61	404	1.85
Shaft Roof, δ_4	508	3.49	508	4.01	508	4.46	517	4.01	508	2.96
Cab Floor, δ_5	607	4.98	603	4.62	601	4.63	610	4.02	607	3.74
Bottom of Windows	706	6.17	706	5.38	701	5.02	708	5.45	706	5.92
Top of Windows	810	7.16	810	6.14	801	5.44	818	10.67	814	11.72
Cab Roof, δ_6	911	7.50	911	6.38	901	5.59	921	12.14	916	13.33

shear wall windows are unrealistically low because of the shear wall window stiffening. These cases are intended for shaft evaluation only, and will produce unrealistic results in the cab. Case L11, without the shear wall windows, shows that the cab will experience significant deflections. These deflections are even greater for the San Carlos tower cab evaluation, which will be discussed in the section for that tower.

Table 11 presents the Salinas tower story drifts. The only level exceeding the acceptable limits is in the cab between the shaft roof and cab floor. Drifts are much greater in the San Carlos cab evaluation and will be discussed in that section of this report.

Table 11. Salinas Tower Story Drift and P-delta Effect Evaluation.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Analysis Case	Joint #	Elastic Defl δ_x (in.)	Calc Story Drift Δ_c (in.)	Gravity Load above Y_x P_x (kips)	Seismic Shear Force V_x (kips)	Stability Coeff θ	Story Drift w/ P-delta Δ_c (in.)
Ground Floor	0			L3	102	0.00					
2nd Floor	134	134	2.68	L3	204	0.35	0.35				
3rd Floor	278	144	2.88	L3	304	1.37	1.02				
4th Floor	422	144	2.88	L3	404	2.83	1.46				
Shaft Roof	582	160	3.2	L3	508	4.46	1.63				
Shaft Roof	582	160	3.2	L1	513	3.43					
Cab Floor	620	38	0.76	L1	607	4.98	1.55	60	115	0.021	1.55
Cab Roof	777	157	3.14								

Shaft Member and Connection Evaluation

Tables 12 through 16 present the force and moment summary for the most critically stressed members and connections. These tables also present the DCRs for each of these critical components. The force, moment, and DCR tables (e.g., Tables 12 through 16) contain the following information in table columns (left to right) as defined below:

- Component = the description of the critical building member or connection being evaluated.
- Drawing/Section # = the building drawing and section number that defines the component being evaluated. When multiple members are being evaluated, the critical member is underlined in the column.
- Member ID/End = the building finite element model member number and joint number for the most critically loaded (in terms of DCR) member or connection of the particular component being evaluated.
- Length = the length in inches when the component being evaluated is a building member.
- Load Type = the description of the type of load for which the forces and moments are determined – i.e., Gravity, horizontal seismic forces, and vertical seismic forces (either 30 or 31.4 percent of gravity).
- P = the axial force in kips where tension is positive.
- V2 or Vy = the shear force in kips applied to the component in the strong axis of the member from which the force was obtained.
- V3 or Vx = the shear force in kips applied to the component in the weak axis of the member from which the force was obtained.
- T = the torsional moment in kip-inches applied to the component from the end of member for which the torsion was obtained.
- M2 or My = the moment in kip-inches applied to the component in the weak axis of the member from which the force was obtained.
- M3 or Mx = the moment in kip-inches applied to the component in the strong axis of the member from which the force was obtained.

- DCR = the demand capacity ratio. The component being evaluated meets the requirements of this life-safety evaluation if the DCR values fall below 2.0.

- App # = the appendix number for the MathCAD files that define the capacity definition for the component being evaluated. The total applied forces and moments presented in these tables are entered into these files to define the DCR for these particular forces and moments. Appendices B1 through B14 define the capacity of all Type L tower components being evaluated using the forces and moments from the L1 analysis only. Tables 13 through 15; 20 through 25; 29 and 34 reference the same Appendices but use different forces and moments to determine their DCRs. Appendix B15 defines the capacity of a structural tube member used in the tower retrofit. Values from this evaluation are only used in the upgrade analysis as presented in Tables 29 and 34. The tables and the report Table of Contents define the particular component for which capacity is determined in each of the appendices.

Case L1 with the full seismic spectrum at 90 degrees to the X-axis (θ_L = 90°) gives the worst loading for critical shaft members and connections. The highest shaft DCR is at the horizontal portion of the column bent, at the bent corner at the roof. This has a DCR of 2.11, which is above 2.0. As was described earlier, DCRs up to 2.0 are acceptable because of the conservative use of an R value of 1.0 in the analysis. The highest DCR of 2.11 is with the full seismic loading at 55.3 degrees and with one window acting as a shear wall (case L4). The conservative assumption of the shear walls acting as fully effective shear walls results in loading the horizontal portion of the bent much more heavily. This influence can be seen by comparing the maximum DCR of this member in the L1 and L11 analyses cases. The L1 and L11 analysis are identical except that the L1 assumes all windows act as shear walls and this yields a DCR of 2.03, whereas L11 assumes no shear walls and this yields a DCR of 1.69. The shear wall assumption is very conservative, and the horizontal portion of the bent, as well as all other members and connections of the shaft, are therefore judged to meet the requirements of this life-safety evaluation. Tables 12 through 16 show that corner mullions and the channels above and below windows have DCRs well above 2.0. These members are in the cab and cab elements will be evaluated in the San Carlos tower evaluation, which is even more critical for cab components.

Table 12. Forces, Moments, and DCRs (L1, 50-ft tower, all windows, θ_L = 90°, S_{DS} = 1.00g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.1	Gravity	-111.3	1.4	-0.1	0	-5	211		
at Base		12		H. Seismic	69.8	93.3	-10.6	862	-3470	37971		
				0.300 Gravity	33.4	0.4	0.0	0	-2	63		
(counteract)		n=102		Total	-8.1	95.1	-10.7	862	-3477	38245	0.96	B1
Bent Column	S1	NPCM	160.1	Gravity	-26.9	8.2	0.0	-2	-15	-817		
at Shaft		42		H. Seismic	69.9	72.4	3.2	-438	-357	-7848		
				0.300 Gravity	8.1	2.5	0.0	-1	-5	-245		
(counteract)		n=536		Total	51.1	83.0	3.2	-440	-377	-8910	1.15	B2
Bent Beam	S1	TBNT	92.9	Gravity	-5.3	-7.4	0.2	1	0	-531		
at Shaft		532		H. Seismic	56.2	-45.6	2.3	381	429	-7600		
				0.300 Gravity	1.6	-2.2	0.0	0	0	-159		
(counteract)		n=508		Total	52.5	-55.2	2.5	382	430	-8290	2.03	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-6.8	0.9	0.1	1	0	0		
at center of		534		H. Seismic	-11.4	44.3	3.2	382	0	0		
tower roof				0.300 Gravity	-2.0	0.3	0.0	0	0	0		
(additive)		n=531		Total	-20.2	45.4	3.3	383	0	0	0.48	B4
Bent/roof	S2	EB	-	Gravity	-2.9	14.6	-0.3	2	0	0		
beam conn	S4/13,19	509		H. Seismic	-24.0	31.9	-4.8	5	0	0		
16WF88	S5/17,18			0.300 Gravity	-0.9	4.4	-0.1	1	0	0		
(additive)		n=514		Total	-27.8	50.8	-5.2	8	0	0	0.59	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-10.5	-0.8	2.4	0	-4	-34		
mullions at		1		H. Seismic	-23.3	-19.7	9.7	54	-242	-670		
their base				0.300 Gravity	-3.2	-0.2	0.7	0	-1	-10		
(additive)	col. 1-4	n=601		Total	-36.9	-20.6	12.8	55	-246	-714	1.13	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-13.7	3.0	0.2	-1	6	144		
mullion/		4		H. Seismic	14.1	20.4	10.1	-68	59	617		
10WF conn				0.300 Gravity	4.1	0.9	0.0	0	2	43		
(counteract)	col. 1-4	n=534		Total	4.5	24.3	10.3	-70	66	804	6.15	B7
Corner	S4/8,9	CLCM	-	Gravity	-8.2	0.8	-2.6	-2	-94	-67		
mullion/		3		H. Seismic	29.3	25.5	-11.6	-46	-288	-207		
10WF conn				0.300 Gravity	2.5	0.2	-0.8	-1	-28	-20		
(counteract)	col. 5	n=506		Total	23.5	26.5	-15.0	-48	-410	-294	2.07	B8
10WF72	S2,S3	RAD	43.4	Gravity	-1.8	16.2	0.1	0	-3	-551		
at 16WF88	S4/A	525		H. Seismic	-27.4	35.6	4.7	22	-230	-2385		
shaft roof				0.300 Gravity	-0.5	4.9	0.0	0	-1	-165		
(additive)		n=515		Total	-29.7	56.6	4.8	22	-233	-3101	1.19	B9
Cab floor	S2	CEPG	63.0	Gravity	2.5	-2.7	0.2	0	-11	191		
12WF27		602		H. Seismic	13.6	-2.3	1.0	0	-65	142		
near Col 5				0.300 Gravity	0.8	-0.8	0.1	0	-3	57		
(additive)		n=604		Total	16.8	-5.8	1.3	0	-79	390	0.73	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-2.1	0.1	-0.1	0	0	6		
at base of		704		H. Seismic	-16.6	2.4	-0.8	0	0	115		
window				0.300 Gravity	-0.6	0.0	0.0	0	0	2		
(additive)		n=704		Total	-19.3	2.5	-0.9	0	0	123	2.98	B11
Interior	A5/5	WM	90.4	Gravity	0.5	0.0	0	0	0	0		
mullion		2		H. Seismic	-16.4	0.0	0	1	0	1		
(S3x7.5)				0.300 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	-16.0	0.0	0	1	0	1	0.90	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-2.1	0.2	-0.3	-5	-6	6		
mullion at		7		H. Seismic	-2.2	9.6	-5.0	-43	-142	257		
the roof				0.300 Gravity	-0.6	0.1	-0.1	-2	-2	2		
(additive)		n=805		Total	-4.9	9.9	-5.3	-50	-150	266	0.50	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.9	-0.7	-0.1	0	0	-27		
at top of		818		H. Seismic	-13.1	-8.5	-5.1	1	0	-352		
window				0.300 Gravity	-0.3	-0.2	0.0	0	0	-8		
(buckling)		n=818		Total	-12.5	-9.4	-5.2	1	0	-387	2.54	B14

Table 13. Forces, Moments, and DCRs (L2, 50-ft tower, all windows, $\theta_L = 45°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.1	Gravity	-111.3	1.4	-0.1	0	-5	211		
at Base		12		H. Seismic	68.4	87.6	-16.3	1454	-4972	35774		
				0.300 Gravity	33.4	0.4	0.0	0	-2	63		
(counteract)		n=102		Total	-9.5	89.5	-16.3	1454	-4979	36048	1.05	B1
Bent Column	S1	NPCM	160.1	Gravity	-26.9	8.2	0.0	-2	-15	-817		
at Shaft		42		H. Seismic	68.4	63.0	4.8	-858	-770	-6956		
				0.300 Gravity	8.1	2.5	0.0	-1	-5	-245		
(counteract)		n=536		Total	49.6	73.7	4.8	-860	-790	-8018	1.14	B2
Bent Beam	S1	TBNT	92.9	Gravity	-4.7	-7.0	0.0	-3	-3	-467		
at Shaft		529		H. Seismic	35.6	-38.5	-10.6	-742	-1431	-6499		
				0.300 Gravity	1.4	-2.1	0.0	-1	-1	-140		
(counteract)		n=501		Total	32.3	-47.6	-10.6	-746	-1435	-7106	2.00	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-6.7	-1.0	0.0	1	0	0		
at center of		538		H. Seismic	-6.1	-41.2	-7.1	727	0	0		
tower roof				0.300 Gravity	-2.0	-0.3	0.0	0	0	0		
(additive)		n=531		Total	-14.8	-42.4	-7.2	727	0	0	0.44	B4
Bent/roof	S2	EB	-	Gravity	-2.9	14.6	-0.3	2	0	0		
beam conn	S4/13,19	509		H. Seismic	-17.4	22.7	-3.3	4	0	0		
16WF88	S5/17,18			0.300 Gravity	-0.9	4.4	-0.1	1	0	0		
(additive)		n=514		Total	-21.2	41.6	-3.7	6	0	0	0.48	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-8.0	0.8	-2.6	-2	4	-98		
mullions at		3		H. Seismic	-28.9	19.5	-12.4	-70	222	-687		
their base				0.300 Gravity	-2.4	0.2	-0.8	-0.6	1.2	-29.5		
(additive)	col. 1-4	n=603		Total	-39.3	20.6	-15.7	-72	227	-814	1.19	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-12.5	2.0	-0.2	1	-6	119		
mullion/		5		H. Seismic	13.0	20.7	-7.2	76	-44	639		
10WF conn				0.300 Gravity	3.7	0.6	-0.1	0	-2	36		
(counteract)	col. 1-4	n=532		Total	4.2	23.4	-7.5	78	-51	793	6.02	B7
Corner	S4/8,9	CLCM	-	Gravity	-8.9	0.8	-2.6	-2	-94	-67		
mullion/		3		H. Seismic	28.9	19.5	-12.4	-70	-469	-344		
10WF conn				0.300 Gravity	2.7	0.2	-0.8	-1	-28	-20		
(counteract)	col. 5	n=506		Total	22.7	20.6	-15.7	-72	-590	-431	2.98	B8
10WF72	S2,S3	RAD	43.4	Gravity	-1.8	16.2	0.1	0	-3	-551		
at 16WF88	S4/A	525		H. Seismic	-19.5	25.3	7.5	31	-346	-1692		
shaft roof				0.300 Gravity	-0.5	4.9	0.0	0	-1	-165		
(additive)		n=515		Total	-21.8	46.3	7.6	31	-350	-2408	1.04	B9
Cab floor	S2	CEPG	63.0	Gravity	2.5	-2.7	0.2	0	-11	191		
12WF27		602		H. Seismic	10.4	-1.4	0.8	0	-51	91		
near Col 5				0.300 Gravity	0.8	-0.8	0.1	0	-3	57		
(additive)		n=604		Total	13.6	-5.0	1.0	1	-64	339	0.61	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-1.1	0.1	-0.1	0	0	2		
at base of		712		H. Seismic	-20.2	1.8	-12.9	0	0	90		
window				0.300 Gravity	-0.3	0.0	0.0	0	0	1		
(additive)		n=712		Total	-21.6	1.9	-13.1	0	0	93	2.77	B11
Interior	A5/5	WM	90.4	Gravity	0.5	0.0	0	0	0	0		
mullion		2		H. Seismic	-11.4	0.0	0	1	0	1		
(S3x7.5)				0.300 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	-11.1	0.0	0	1	0	1	0.63	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-2.5	0.1	0.3	4	8	2		
mullion at		6		H. Seismic	-2.5	11.0	5.5	37	156	293		
the roof				0.300 Gravity	-0.7	0.0	0.1	1	2	1		
(additive)		n=801		Total	-5.7	11.1	6.0	43	166	296	0.56	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.8	0.6	0.1	0	0	21		
at top of		808		H. Seismic	-10.2	6.3	3.8	0	0	248		
window				0.300 Gravity	-0.2	0.2	0.0	0	0	6		
(buckling)		n=808		Total	-9.7	7.0	4.0	0	0	275	1.74	B14

Table 14. Forces, Moments, and DCRs (L3, 50-ft tower, all windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.1	Gravity	-119.1	1.3	0.1	-2	20	-1		
at Base		11		H. Seismic	73.52	92.2	19.9	-1656	5782	-37365		
				0.300 Gravity	35.7	0.4	0.0	0	6	0		
(counteract)		n=101		Total	-9.9	93.8	20.1	-1658	5808	-37366	1.15	B1
Bent Column	S1	NPCM	160.1	Gravity	-30.0	7.6	-0.1	3	19	-800		
at Shaft		41		H. Seismic	73.6	61.2	-5.8	1704	938	-6856		
				0.300 Gravity	9.0	2.3	0.0	1	6	-240		
(counteract)		n=535		Total	52.6	71.0	-5.9	1709	963	-7896	1.17	B2
Bent Beam	S1	TBNT	92.9	Gravity	-4.7	-7.0	0.0	-3	-3	-467		
at Shaft		529		H. Seismic	26.8	-37.7	11.9	-836	-1632	-6466		
				0.300 Gravity	1.4	-2.1	0.0	-1	-1	-140		
(counteract)		n=501		Total	23.5	-46.8	11.9	-840	-1635	-7073	2.03	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-5.1	-4.0	-0.1	-3	-4	-144		
at center of		536		H. Seismic	-6.4	-42.2	-7.6	-834	0	0		
tower roof				0.300 Gravity	-1.5	-1.2	0.0	-1	-1	-43		
(additive)		n=531		Total	-13.0	-47.4	-7.7	-838	-5	-188	0.50	B4
Bent/roof	S2	EB	-	Gravity	-2.9	-14.9	0.3	-2	0	0		
beam conn	S4/13,19	510		H. Seismic	-9.5	-12.5	2.1	-3	0	0		
16WF88	S5/17,18			0.300 Gravity	-0.9	-4.5	0.1	-1	0	0		
(additive)		n=516		Total	-13.2	-31.9	2.4	-5	0	0	0.37	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-8.0	0.8	-2.6	-2	4	-98		
mullions at		3		H. Seismic	-31.2	14.9	-12.4	-81	146	-402		
their base				0.300 Gravity	-2.4	0.2	-0.8	-0.6	1.2	-29.5		
(additive)	col. 1-4	n=603		Total	-41.5	16.0	-15.8	-84	151	-530	0.80	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-10.7	-0.8	2.4	0	88	-63		
mullion/		1		H. Seismic	32.9	-16.6	13.6	68	485	-359		
10WF conn				0.300 Gravity	3.2	-0.2	0.7	0	26	-19		
(counteract)	col. 1-4	n=503		Total	25.5	-17.6	16.7	68	599	-441	6.51	B7
Corner	S4/8,9	CLCM	-	Gravity	-8.2	0.8	-2.6	-2	-94	-67		
mullion/		3		H. Seismic	31.2	14.9	-12.4	-81	-542	-400		
10WF conn				0.300 Gravity	2.5	0.2	-0.8	-1	-28	-20		
(counteract)	col. 5	n=506		Total	25.4	16.0	-15.8	-84	-664	-487	3.42	B8
10WF72	S2,S3	RAD	43.4	Gravity	-1.8	16.2	0.1	0	-3	-551		
at 16WF88	S4/A	525		H. Seismic	-9.7	14.2	8.5	33	-385	-903		
shaft roof				0.300 Gravity	-0.5	4.9	0.0	0	-1	-165		
(additive)		n=515		Total	-12.0	35.3	8.6	33	-388	-1619	0.81	B9
Cab floor	S2	CEPG	63.0	Gravity	2.5	-2.7	0.2	0	-11	191		
12WF27		602		H. Seismic	6.9	-1.7	0.5	0	-33	108		
near Col 5				0.300 Gravity	0.8	-0.8	0.1	0	-3	57		
(additive)		n=604		Total	10.2	-5.3	0.7	0	-46	356	0.49	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-1.1	0.1	-0.1	0	0	2		
at base of		712		H. Seismic	-20.7	1.3	-7.7	-1	0	68		
window				0.300 Gravity	-0.3	0.0	0.0	0	0	1		
(additive)		n=712		Total	-22.1	1.4	-7.9	-1	0	70	2.33	B11
Interior	A5/5	WM	90.4	Gravity	-0.2	0.0	0	0	0	0		
mullion		5		H. Seismic	-6.9	0.0	0	-1	0	0		
(S3x7.5)				0.300 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=712		Total	-7.1	0.0	0	-1	0	0	0.41	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-2.1	0.2	-0.3	-5	-6	-6		
mullion at		7		H. Seismic	-3.0	11.7	-5.2	-39	-146	-311		
the roof				0.300 Gravity	-0.6	0.1	-0.1	-2	-2	-2		
(additive)		n=805		Total	-5.8	11.9	-5.5	-46	-154	-320	0.56	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.9	-0.6	-0.1	0	0	25		
at top of		819		H. Seismic	-7.7	-4.0	-2.5	0	0	159		
window				0.300 Gravity	-0.3	-0.2	0.0	0	0	7		
(buckling)		n=820		Total	-7.0	-4.8	-2.6	0	0	191	1.23	B14

Table 15. Forces, Moments, and DCRs (L4, 50-ft tower, one window, θ_L = 55.3°, S_{DS} = 1.00g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.1	Gravity	-119.3	1.3	0.1	-2	20	-3		
at Base		11		H. Seismic	49.1	106.9	18.8	-1548	4503	-41335		
				0.300 Gravity	35.8	0.4	0.0	0	6	-1		
(counteract)		101		Total	-34.5	108.5	19.0	-1550	4529	-41338	1.08	B1
Bent Column	S1	NPCM	160.1	Gravity	-30.2	7.3	-0.1	4	19	-770		
at Shaft		41		H. Seismic	48.9	61.0	-5.2	1697	835	-7020		
				0.300 Gravity	9.1	2.2	0.0	1	6	-231		
(counteract)		n=535		Total	27.7	70.5	-5.2	1702	859	-8021	1.12	B2
Bent Beam	S1	TBNT	92.9	Gravity	-4.3	-6.7	0.0	-3	-3	-432		
at Shaft		529		H. Seismic	25.5	-41.0	11.3	-811	-1615	-6911		
				0.300 Gravity	1.3	-2.0	0.0	-1	-1	-130		
(counteract)		n=501		Total	22.5	-49.8	11.3	-815	-1619	-7472	2.11	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-4.5	-1.2	-0.1	-3	0	0		
at center of		536		H. Seismic	-19.3	-41.0	-7.7	-810	0	0		
tower roof				0.300 Gravity	-1.3	-0.4	0.0	-1	0	0		
(additive)		n=531		Total	-25.1	-42.6	-7.8	-814	0	0	0.45	B4
Bent/roof	S2	EB	-	Gravity	-2.6	-14.7	0.1	-2	0	0		
beam conn	S4/13,19	510		H. Seismic	-13.8	-7.6	2.5	-4	0	0		
16WF88	S5/17,18			0.300 Gravity	-0.8	-4.4	0.0	-1	0	0		
(additive)		n=516		Total	-17.2	-26.6	2.6	-7	0	0	0.31	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-13.7	3.1	-0.3	-4	19	-9		
mullions at		4		H. Seismic	-4.0	17.2	-7.0	-128	294	-468		
their base				0.300 Gravity	-4.1	0.9	-0.1	-1.3	5.6	-2.6		
(additive)	col. 1-4	n=610		Total	-21.8	21.3	-7.4	-133	318	-479	1.01	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-13.3	1.4	-0.4	-1	-7	115		
mullion/		5		H. Seismic	19.9	23.0	-3.5	-127	-41	1281		
10WF conn				0.300 Gravity	4.0	0.4	-0.1	0	-2	34		
(counteract)	col. 1-4	n=532		Total	10.6	24.8	-4.0	-128	-50	1430	12.7	B7
Corner	S4/8,9	CLCM	-	Gravity	-8.2	0.4	-2.0	-15	-53	-36		
mullion/		3		H. Seismic	17.4	5.6	-17.7	-13	-806	-584		
10WF conn				0.300 Gravity	2.5	0.1	-0.6	-4	-16	-11		
(counteract)	col. 5	n=506		Total	11.7	6.1	-20.2	-32	-874	-631	4.08	B8
10WF72	S2,S3	RAD	30.8	Gravity	-1.4	13.5	-0.4	-7	10	-299		
at 16WF88	S4/A	523		H. Seismic	-23.0	19.9	-3.6	-41	99	-1365		
shaft roof				0.300 Gravity	-0.4	4.1	-0.1	-2	3	-90		
(additive)		n=511		Total	-24.8	37.5	-4.0	-50	111	-1754	0.67	B9
Cab floor	S2	CEPG	63.0	Gravity	1.7	-2.7	0.2	0	-14	191		
12WF27		602		H. Seismic	11.8	-1.5	0.5	1	-31	95		
near Col 5				0.300 Gravity	0.5	-0.8	0.1	0	-4	57		
(additive)		n=604		Total	14.0	-5.1	0.8	1	-49	343	0.50	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-1.0	0.0	-0.3	0	0	-5		
at base of		703		H. Seismic	-7.1	-2.0	-12.4	0	0	-106		
window				0.300 Gravity	-0.3	0.0	0.9	1	0	-1		
(additive)		n=703		Total	-8.3	-2.1	-11.8	1	0	-112	1.55	B11
Interior	A5/5	WM	90.4	Gravity	-0.6	0.0	0	0	0	0		
mullion		2		H. Seismic	-11.0	0.0	0	-2	0	0		
(S3x7.5)				0.300 Gravity	-0.2	0.0	0	0	0	0		
(buckling)		n=807		Total	-11.7	0.0	0	-2	0	0	0.65	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-5.8	0.3	-0.5	-9	-10	7		
mullion at		9		H. Seismic	-9.2	10.1	-7.6	-108	-185	269		
the roof				0.300 Gravity	-1.7	0.1	-0.1	-3	-3	2		
(additive)		n=810		Total	-16.8	10.4	-8.1	-119	-198	278	0.61	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	1.7	0.7	0.2	0	0	-29		
at top of		809		H. Seismic	-15.6	6.9	3.9	1	0	-285		
window				0.300 Gravity	-0.5	0.2	0.1	0	0	-9		
(buckling)		n=810		Total	-14.4	7.8	4.2	1	0	-323	2.27	B14

Table 16. Forces, Moments, and DCRs (L11, 50-ft tower, no windows, θ_L = 90°, S_{DS} = 1.00g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.1	Gravity	-111.5	1.5	-0.1	0	-5	201		
at Base		12		H. Seismic	37.8	103.3	-6.9	537	-2430	40165		
				0.300 Gravity	33.5	0.4	0.0	0	-1	60		
(counteract)		n=102		Total	-40.2	105.2	-7.0	538	-2437	40427	0.87	B1
Bent Column	S1	NPCM	160.1	Gravity	-36.4	7.0	0.0	2	4	-652		
at Shaft		44		H. Seismic	35.9	56.8	-2.5	329	269	-6827		
				0.300 Gravity	10.9	2.1	0.0	1	1	-196		
(counteract)		n=538		Total	10.4	65.9	-2.5	332	274	-7674	0.93	B2
Bent Beam	S1	TBNT	92.9	Gravity	-5.7	-3.6	0.0	1	1	-248		
at Shaft		537		H. Seismic	17.5	-43.6	-1.6	291	348	-6865		
				0.300 Gravity	1.7	-1.1	0.0	0	0	-74		
(counteract)		n=517		Total	13.5	-48.3	-1.6	292	350	-7188	1.69	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-5.5	-1.4	0.0	1	0	0		
at center of		538		H. Seismic	-14.8	-38.4	3.0	291	0	0		
tower roof				0.300 Gravity	-1.6	-0.4	0.0	0	0	0		
(additive)		n=531		Total	-22.0	-40.3	3.1	292	0	0	0.42	B4
Bent/roof	S2	EB	-	Gravity	-1.9	16.1	-0.4	1	0	0		
beam conn	S4/13,19	511		H. Seismic	-15.9	13.7	-4.4	3	0	0		
16WF88	S5/17,18			0.300 Gravity	-0.6	4.8	-0.1	0	0	0		
(additive)		n=518		Total	-18.4	34.7	-4.9	4	0	0	0.41	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-15.0	-0.5	0.1	3	-1	-31		
mullions at		6		H. Seismic	-6.5	-23.1	2.4	92	-39	-1657		
their base				0.300 Gravity	-4.5	-0.2	0.0	1	0	-9		
(additive)	col. 1-4	n=533		Total	-26.1	-23.8	2.6	95	-40	-1697	1.67	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-15.0	-0.5	0.1	3	-1	-31		
mullion/		6		H. Seismic	6.5	-23.1	2.4	92	-39	-1657		
10WF conn				0.300 Gravity	4.5	-0.2	0.0	1	0	-9		
(counteract)	col. 1-4	n=533		Total	-4.0	-23.8	2.6	95	-40	-1697	10.9	B7
Corner	S4/8,9	CLCM	-	Gravity	-9.0	-0.2	-1.6	0	-72	-49		
mullion/		3		H. Seismic	4.7	-9.3	-9.5	109	-354	-262		
10WF conn				0.300 Gravity	2.7	-0.1	-0.5	0	-22	-15		
(counteract)	col. 5	n=506		Total	-1.7	-9.6	-11.6	109	-447	-326	1.92	B8
10WF72	S2,S3	RAD	43.4	Gravity	0.5	15.3	0.1	-1	-2	-688		
at 16WF88	S4/A	525		H. Seismic	23.1	6.6	2.5	-39	-91	-1857		
shaft roof				0.300 Gravity	0.2	4.6	0.0	0	-1	-207		
(additive)		n=515		Total	23.8	26.5	2.6	-40	-93	-2752	0.98	B9
Cab floor	S2	CEPG	63.0	Gravity	1.1	-2.7	0.3	0	-18	191		
12WF27		602		H. Seismic	15.6	-1.4	1.0	1	-62	86		
near Col 5				0.300 Gravity	0.3	-0.8	0.1	0	-5	57		
(additive)		n=604		Total	17.0	-4.9	1.4	1	-85	335	0.69	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-2.9	0.0	-0.1	0	0	-4		
at base of		701		H. Seismic	-6.9	0.3	0.0	0	0	-30		
window				0.300 Gravity	-0.9	0.0	0.0	0	0	-1		
(additive)		n=702		Total	-10.6	0.3	-0.1	0	0	-36	0.79	B11
Interior	A5/5	WM	90.4	Gravity	0.3	0.0	0.0	0	0	0		
mullion		2		H. Seismic	-0.5	0.0	0.0	0	0	-2		
(S3x7.5)				0.300 Gravity	-0.1	0.0	0.0	0	0	0		
(buckling)		n=704		Total	-0.3	0.1	0.0	0	0	-3	0.05	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-5.8	0.9	0.5	1	14	24		
mullion at		9		H. Seismic	-3.2	4.8	7.3	76	184	129		
the roof				0.300 Gravity	-1.7	0.3	0.2	0	4	7		
(additive)		n=810		Total	-10.7	6.0	8.0	77	202	160	0.50	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.0	0.1	0.0	0	0	15		
at top of		808		H. Seismic	-8.6	1.7	0.3	0	0	140		
window				0.300 Gravity	0.0	0.0	0.0	0	0	4		
(buckling)		n=808		Total	-8.6	1.9	0.3	0	0	159	1.05	B14

San Carlos Tower – Cab Members and Connections

The Type L tower cab members and connections were most critical at the San Carlos tower. This tower cab was evaluated with analysis Cases L5 through L10, and L12 (see Table 6 for a summary). The most critical case was L6. Case L7 is for one window acting as a shear wall and cases L8 through L10 are for all the windows acting as shear walls. Only deflection results are presented for Cases L9 and L10, because these were never a critical load condition. The greatest problem in the cab evaluation is the excessive story drifts. The following sections summarize the Type L cab evaluations in terms of modal behavior, deflections, story drifts, and component forces and moments used to calculate DCRs.

Modal Analysis Results and Deflections

Table 17 gives the primary modes of vibration for the Type L, Cases L5 and L6 evaluations. Table 18 gives the maximum deflections at each floor level and other key locations in the San Carlos tower. Note that the deflections above the shaft roof become excessive. These are greater than any of the Salinas tower cases. Table 19 gives the greatest San Carlos story drifts, which are always for Case L6 in the cab. The calculated story drifts are the difference in SRSS deflections between floor levels (e.g., cab floor and roof). Table 19 shows that the story drift between the cab floor and roof is 14.14 in., which is more than four times greater than acceptable drifts calculated based on FEMA 302 guidance (Equation 24 of this report). These excessive drifts are due to very large rotations at the connections of the mullion base to the supporting beams at the shaft roof. The supporting shaft roof members provide little resistance to rotation. When seismic loads are applied along the Y-axis, four of the wide flange beams supporting the mullions are loaded in torsion, and these sections have very low torsional stiffness. These open sections (16WF40 and 10WF72) have very small polar moments of inertia, J. The fifth mullion is supported at the end of a cantilevered wide flange beam, and this also provides little rotational resistance. Connections of the mullions at the cab floor and roof are pinned (also connections above and below the windows), so that the only overturning resistance for the mullions is the very flexible moment connection at the mullion base. In reality, the shear connections at the cab floor and roof levels will provide some moment resistance, and this could have been modeled by adding rotational spring elements to the SAP2000 models. Even a very small amount of moment resistance at these joints greatly reduces deflections and explains why large deflections in the cab do not occur during strong winds or low seismic motions. However, this moment resistance should not properly be accounted for in the design or evaluation of these towers, and they remain very vulnerable to excessive deflections. This is the greatest Type L vulnerability that must be eliminated in upgrade development.

Table 17. San Carlos (Type L – 30 ft) Cab Evaluation Modal Analysis Results.

Mode #	L5 and L6: Period (sec)	L5 and L6: Cumulative Participating Mass (%) X-dir	Y-dir	Z-dir	Mode of Vibration	L7 Period (sec)
1	1.317	5.0	6.0	0.0	1st YX-Lat. & Torsion/cab	1.307
2	1.273	8.3	15.5	0.1	1st Y-Lateral/cab	0.973
3	0.776	14.4	15.5	0.1	1st X-Lateral & Torsion/cab	0.448
6	0.271	15.4	15.6	0.1	2nd Torsion	0.255
9	0.208	15.5	22.0	13.5	1st Vertical & Y-Lateral	0.208
10	0.195	15.8	39.5	16.2	2nd Y-Lateral	
11	0.189	45.4	39.6	16.3	2nd X-Lateral	0.187
13	0.176	52.0	39.6	16.3	2nd X-Lateral/windows	0.175
14	0.172	52.1	53.3	16.8	2nd Y-Lateral/windows	0.167
15	0.151	52.2	53.9	17.0	Vertical/X-cab rocking	0.144
20	0.134	71.4	56.4	18.7	3rd X-Lateral	0.127
21	0.128	72.1	57.0	20.8	Vertical/Y-cab rocking	0.128
22	0.122	72.5	71.8	20.9	3rd Y-Lateral	0.097
30	0.097	73.4	76.9	31.7	3rd Vertical/2nd floor	0.097
34	0.089	73.5	76.9	42.6	4th Vertical/shaft roof	0.089
49	0.062	77.2	77.1	47.4	3rd Torsion	0.061
63	0.037	81.7	93.2	50.0	4th Y-Lateral	0.037
64	0.035	94.1	96.3	50.0	4th X-Lateral	0.035

Table 18. San Carlos Cab Evaluation Selected Horizontal SRSS Deflections.

Location	L5 Joint #	L5 δ_{xe} (in.)	L6 Joint #	L6 δ_{xe} (in.)	L7 Joint #	L7 δ_{xe} (in.)	L8 Joint #	L8 δ_{xe} (in.)	L9 Joint #	L9 δ_{xe} (in.)	L10 Joint #	L10 δ_{xe} (in.)
2nd Floor, δ_1	201	0.09	209	0.06	204	0.11	209	0.10	201	0.14	201	0.16
Shaft Roof, δ_2	501	0.34	513	0.24	508	0.42	513	0.38	501	0.51	501	0.60
Cab Floor, δ_3	609	2.49	609	3.11	609	2.72	607	1.56	609	1.64	609	1.39
Bottom of Windows	708	5.48	708	6.93	710	6.00	706	2.51	708	2.47	708	2.10
Top of Windows	814	11.93	814	15.13	818	13.18	810	3.26	814	3.13	814	2.63
Cab Roof, δ_4	916	13.58	916	17.25	921	14.97	911	3.51	916	3.32	916	2.78

Table 19. San Carlos Tower Story Drift and P-delta Effect Evaluation.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Analysis Case	Joint #	Elastic Defl δ_x (in.)	Calc Story Drift Δ_c (in.)	Gravity Load above Y_x P_x (kips)	Seismic Shear Force V_x (kips)	Stability Coeff θ	Story Drift w/ P-delta Δ_c (in.)
Ground Floor	0			L5	101	0.00					
2nd Floor	134	134	2.68	L5	201	0.09	0.09				
2nd Floor	134	134	2.68	L6	209	0.06					
Shaft Roof	294	160	3.2	L6	513	0.24	0.18				
Cab Floor	332	38	0.76	L6	609	3.11	2.87	60	100	0.046	2.87
Cab Roof	489	157	3.14	L6	916	17.25	14.14	26	63	0.038	14.14

Cab Member and Connection Evaluation

Tables 20 through 23 present the force and moment summary for the most critically stressed cab members and connections. These tables also present DCRs for each of these critical components. Case L6 with the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90°$) gives the worst DCRs for cab members and connections. The mullion base plate connection fails with a very high DCR of 14.3. This failure is due to bending in the base plate for the smaller mullions (Col 1–4 on Drawing S3; Corlett & Spackman 1966). All other DCRs are much smaller. This failure was further evaluated by determining the seismic factor, F_1, at which all mullion connections (including the heavier Col 5) have a DCR of 1.0. Table 24 summarizes the forces, moments, and DCRs for all the mullion connections to the tower shaft. The heavy Col 5 mullion connection fails by tension in the bolts. Table 25 shows the forces and moments at a seismic factor, F_1 equal to 0.1055, which gives an average DCR of 1.0. At this load condition, all the base plates will begin to yield (or bolt yielding for Col 5), and a hinge will form at each connection. This will cause a collapse mechanism, because no other connection to the mullions prevents mullion overturning. In reality, the shear connections will carry some moment so that yielding of the plates does not occur until a greater seismic factor. But these shear connectors will certainly not prevent collapse of the tower cab. The same upgrade that decreases story drift must also correct this connection failure.

Table 20. Forces, Moments, and DCRs (L5, 30-ft tower, no windows, $\theta_L = 0°$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.3	Gravity	-65.5	-2.1	0.1	3	16	21		
at Base		32		H. Seismic	15.6	-36.6	6.8	537	1515	8590		
				0.314 Gravity	20.6	-0.7	0.0	1	5	7		
(counteract)		n=118		Total	-29.3	-39.4	6.9	541	1536	8618	0.40	B1
Bent Column	S1	NPCM	160.2	Gravity	-27.3	8.4	0.0	-1	-17	-724		
at Shaft		42		H. Seismic	15.5	32.8	5.9	-286	-225	-1114		
				0.314 Gravity	8.6	2.6	0.0	0	-5	-227		
(counteract)		n=536		Total	-3.2	43.8	5.9	-288	-247	-2065	0.38	B2
Bent Beam	S1	TBNT	92.9	Gravity	-6.3	-6.8	0.1	1	0	-438		
at Shaft		532		H. Seismic	19.7	-8.2	2.2	241	334	-1324		
				0.314 Gravity	2.0	-2.1	0.0	0	0	-137		
(counteract)		n=508		Total	15.4	-17.1	2.3	242	335	-1899	0.56	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-4.8	-1.3	-0.1	-3	0	0		
at center of		536		H. Seismic	-16.3	-10.1	-2.9	-243	0	0		
tower roof				0.314 Gravity	-1.5	-0.4	0.0	-1	0	0		
(additive)		n=531		Total	-22.6	-11.9	-3.0	-246	0	0	0.32	B4
Bent/roof	S2	EB	-	Gravity	-2.4	14.1	0.3	-1	0	0		
beam conn	S4/13,19	508		H. Seismic	-10.2	13.1	5.4	-2	0	0		
16WF88	S5/17,18			0.314 Gravity	-0.7	4.4	0.1	0	0	0		
(additive)		n=512		Total	-13.3	31.7	5.8	-4	0	0	0.37	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-8.9	-0.2	-1.6	0	-12	-41		
mullions at		3		H. Seismic	-4.3	-6.1	-11.7	-164	-429	-580		
their base				0.314 Gravity	-2.8	-0.1	-0.5	0	-4	-13		
(additive)	col. 1-4	n=603		Total	-16.0	-6.4	-13.9	-164	-445	-634	1.35	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-12.5	1.5	0.0	3	-11	84		
mullion/		5		H. Seismic	6.3	24.1	-5.9	131	-56	1323		
10WF conn				0.314 Gravity	3.9	0.5	0.0	1	-3	26		
(counteract)	col. 1-4	n=532		Total	-2.3	26.1	-6.0	136	-70	1434	9.49	B7
Corner	S4/8,9	CLCM	-	Gravity	-9.0	-0.2	-1.6	0	-73	-49		
mullion/		3		H. Seismic	4.3	-6.1	-11.7	-164	-823	-599		
10WF conn				0.314 Gravity	2.8	-0.1	-0.5	0	-23	-16		
(counteract)	col. 5	n=506		Total	-1.9	-6.4	-13.9	-164	-918	-664	3.93	B8
10WF72	S2,S3	RAD	43.4	Gravity	0.5	15.3	0.1	-1	-2	-690		
at 16WF88	S4/A	525		H. Seismic	21.0	5.4	5.1	-46	-210	-1559		
shaft roof				0.314 Gravity	0.2	4.8	0.0	0	-1	-217		
(additive)		n=515		Total	21.6	25.5	5.3	-47	-213	-2465	0.96	B9
Cab floor	S2	CEPG	63.0	Gravity	1.1	-2.7	0.3	0	-18	191		
12WF27		602		H. Seismic	10.2	-1.1	0.8	1	-49	72		
near Col 5				0.314 Gravity	0.4	-0.9	0.1	0	-6	60		
(additive)		n=604		Total	11.7	-4.7	1.2	1	-74	323	0.57	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-1.9	0.0	-0.1	0	0	-4		
at base of		708		H. Seismic	-5.0	1.2	0.0	0	0	-113		
window				0.314 Gravity	-0.6	0.0	0.0	0	0	-1		
(additive)		n=709		Total	-7.5	1.2	-0.1	0	0	-119	1.57	B11
Interior	A5/5	WM	90.4	Gravity	0.4	0.0	0	0	0	0		
mullion		4		H. Seismic	-0.5	0.0	0	0	0	-2		
(S3x7.5)				0.314 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=816		Total	-0.2	0.1	0	0	0	-3	0.05	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-5.3	-0.6	-0.3	0	-7	-15		
mullion at		6		H. Seismic	-5.1	-8.6	-7.6	95	-188	-231		
the roof				0.314 Gravity	-1.7	-0.2	-0.1	0	-2	-5		
(additive)		n=801		Total	-12.0	-9.4	-8.0	95	-197	-251	0.57	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	-0.5	-0.2	0.0	0	0	18		
at top of		819		H. Seismic	-13.0	-1.7	-0.3	0	0	139		
window				0.314 Gravity	-0.2	-0.1	0.0	0	0	5		
(buckling)		n=820		Total	-13.7	-1.9	-0.3	0	0	162	1.25	B14

Table 21. Forces, Moments, and DCRs (L6, 30-ft tower, no windows, $\theta_L = 90°$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.3	Gravity	-74.6	-0.2	0.1	1	2	121		
at Base		33		H. Seismic	18.3	-35.0	4.1	342	995	7148		
				0.314 Gravity	23.4	0.0	0.0	0	1	38		
(counteract)		n=119		Total	-32.9	-35.2	4.3	343	998	7306	0.29	B1
Bent Column	S1	NPCM	160.2	Gravity	-27.3	8.4	0.0	-1	-17	-724		
at Shaft		42		H. Seismic	12.9	30.7	3.9	-193	-159	-1303		
				0.314 Gravity	8.6	2.6	0.0	0	-5	-227		
(counteract)		n=536		Total	-5.9	41.8	4.0	-194	-181	-2254	0.35	B2
Bent Beam	S1	TBNT	92.9	Gravity	-6.3	-6.8	0.1	1	0	-438		
at Shaft		532		H. Seismic	22.8	-9.3	2.0	162	229	-1495		
				0.314 Gravity	2.0	-2.1	0.0	0	0	-137		
(counteract)		n=508		Total	18.5	-18.2	2.1	163	230	-2070	0.58	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-6.3	-1.6	0.0	1	0	0		
at center of		538		H. Seismic	-18.2	-11.3	2.2	164	0	0		
tower roof				0.314 Gravity	-2.0	-0.5	0.0	0	0	0		
(additive)		n=531		Total	-26.5	-13.3	2.2	165	0	0	0.37	B4
Bent/roof	S2	EB	-	Gravity	-2.2	16.3	-0.4	1	0	0		
beam conn	S4/13,19	511		H. Seismic	-7.5	16.9	-5.8	3	0	0		
16WF88	S5/17,18			0.314 Gravity	-0.7	5.1	-0.1	0	0	0		
(additive)		n=518		Total	-10.3	38.3	-6.3	5	0	0	0.45	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-15.1	-0.5	0.1	3	-1	-31		
mullions at		6		H. Seismic	-12.0	-28.5	2.6	129	-56	-1954		
their base				0.314 Gravity	-4.7	-0.2	0.0	1	0	-10		
(additive)	col. 1-4	n=533		Total	-31.8	-29.2	2.8	133	-57	-1994	1.98	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-15.1	-0.5	0.1	3	-1	-31		
mullion/		6		H. Seismic	12.0	-28.5	2.6	129	-56	-1954		
10WF conn				0.314 Gravity	4.7	-0.2	0.0	1	0	-10		
(counteract)	col. 1-4	n=533		Total	1.7	-29.2	2.8	133	-57	-1994	14.33	B7
Corner	S4/8,9	CLCM	-	Gravity	-9.0	-0.2	-1.6	0	-73	-49		
mullion/		3		H. Seismic	6.1	-8.2	-8.3	-142	-314	-230		
10WF conn				0.314 Gravity	2.8	-0.1	-0.5	0	-23	-16		
(counteract)	col. 5	n=506		Total	-0.1	-8.5	-10.4	-142	-409	-295	1.77	B8
10WF72	S2,S3	RAD	43.4	Gravity	0.5	15.3	0.1	-1	-2	-690		
at 16WF88	S4/A	525		H. Seismic	28.5	12.1	2.6	-56	-86	-2134		
shaft roof				0.314 Gravity	0.2	4.8	0.0	0	-1	-217		
(additive)		n=515		Total	29.2	32.2	2.8	-57	-88	-3040	1.09	B9
Cab floor	S2	CEPG	63.0	Gravity	1.1	-2.7	0.3	0	-18	191		
12WF27		602		H. Seismic	13.2	-1.9	1.0	1	-62	119		
near Col 5				0.314 Gravity	0.4	-0.9	0.1	0	-6	60		
(additive)		n=604		Total	14.7	-5.5	1.4	1	-87	370	0.66	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-2.9	0.0	-0.1	0	0	-4		
at base of		701		H. Seismic	-7.3	1.2	0.0	0	0	-111		
window				0.314 Gravity	-0.9	0.0	0.0	0	0	-1		
(additive)		n=702		Total	-11.0	1.2	-0.1	0	0	-117	1.84	B11
Interior	A5/5	WM	90.4	Gravity	0.3	0.0	0.0	0	0	0		
mullion		2		H. Seismic	-0.2	0.0	0.0	0	0	-2		
(S3x7.5)				0.314 Gravity	-0.1	0.0	0.0	0	0	0		
(buckling)		n=807		Total	0.0	0.1	0.0	0	0	-3	0.04	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-4.4	-1.0	0.3	4	6	-27		
mullion at		7		H. Seismic	-7.1	-11.7	6.6	143	147	-312		
the roof				0.314 Gravity	-1.4	-0.3	0.1	1	2	-8		
(additive)		n=805		Total	-12.9	-13.0	6.9	148	155	-348	0.59	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.0	0.1	0.0	0	0	15		
at top of		808		H. Seismic	-9.3	1.9	0.3	0	0	153		
window				0.314 Gravity	0.0	0.0	0.0	0	0	5		
(buckling)		n=808		Total	-9.3	2.1	0.3	0	0	173	1.15	B14

Table 22. Forces, Moments, and DCRs (L7, 30-ft tower, one window, $\theta_L = 55.3°$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.3	Gravity	-75.0	-0.2	0.1	1	1	132		
at Base		33		H. Seismic	21.7	-41.9	9.0	799	1953	9178		
				0.314 Gravity	23.6	-0.1	0.0	0	0	41		
(counteract)		n=119		Total	-29.8	-42.2	9.1	801	1954	9351	0.48	B1
Bent Column	S1	NPCM	160.2	Gravity	-30.7	7.8	0.0	1	20	-755		
at Shaft		41		H. Seismic	15.3	33.4	-7.9	512	333	-1037		
				0.314 Gravity	9.6	2.4	0.0	0	6	-237		
(counteract)		n=535		Total	-5.7	43.6	-7.9	514	359	-2030	0.37	B2
Bent Beam	S1	TBNT	92.9	Gravity	-6.2	-7.1	0.2	0	3	-480		
at Shaft		532		H. Seismic	21.4	-6.8	2.4	374	474	-997		
				0.314 Gravity	1.9	-2.2	0.1	0	1	-151		
(counteract)		n=508		Total	17.2	-16.1	2.6	375	478	-1628	0.54	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-5.0	-1.2	0.0	-3	0	0		
at center of		536		H. Seismic	-19.6	-11.7	-4.2	-374	0	0		
tower roof				0.314 Gravity	-1.6	-0.4	0.0	-1	0	0		
(additive)		n=531		Total	-26.2	-13.3	-4.3	-377	0	0	0.37	B4
Bent/roof	S2	EB	-	Gravity	-2.7	14.7	0.3	-1	0	0		
beam conn	S4/13,19	508		H. Seismic	-9.3	20.4	6.0	-3	0	0		
16WF88	S5/17,18			0.314 Gravity	-0.8	4.6	0.1	0	0	0		
(additive)		n=512		Total	-12.8	39.6	6.5	-5	0	0	0.47	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-13.9	3.2	-0.3	-4	8	112		
mullions at		4		H. Seismic	-7.6	19.0	-4.5	-137	76	1097		
their base				0.314 Gravity	-4.4	1.0	-0.1	-1	3	35		
(additive)	col. 1-4	n=506		Total	-25.9	23.2	-4.9	-142	87	1244	1.33	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-13.3	1.4	-0.4	-1	-7	116		
mullion/		5		H. Seismic	15.9	28.8	-4.0	-149	-38	1594		
10WF conn				0.314 Gravity	4.2	0.4	-0.1	0	-2	36		
(counteract)	col. 1-4	n=532		Total	6.7	30.6	-4.5	-150	-48	1746	14.06	B7
Corner	S4/8,9	CLCM	-	Gravity	-8.2	0.4	-2.0	-15	-53	-35		
mullion/		3		H. Seismic	13.8	4.4	-11.5	-150	-571	-416		
10WF conn				0.314 Gravity	2.6	0.1	-0.6	-5	-17	-11		
(counteract)	col. 5	n=506		Total	8.2	4.9	-14.1	-170	-641	-463	2.92	B8
10WF72	S2,S3	RAD	30.8	Gravity	-1.4	13.5	-0.4	-7	10	-298		
at 16WF88	S4/A	523		H. Seismic	-28.8	15.9	-4.1	-37	125	-1712		
shaft roof				0.314 Gravity	-0.4	4.2	-0.1	-2	3	-93		
(additive)		n=511		Total	-30.6	33.7	-4.5	-47	138	-2103	0.80	B9
Cab floor	S2	CEPG	63.0	Gravity	1.7	-2.7	0.2	0	-14	191		
12WF27		602		H. Seismic	8.6	-1.4	0.6	1	-39	85		
near Col 5				0.314 Gravity	0.5	-0.9	0.1	0	-4	60		
(additive)		n=603		Total	10.9	-5.0	0.9	1	-57	336	0.52	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-1.8	0.0	-0.1	0	0	-4		
at base of		706		H. Seismic	-7.0	1.2	0.0	0	0	-115		
window				0.314 Gravity	-0.6	0.0	0.0	0	0	-1		
(additive)		n=707		Total	-9.3	1.2	-0.1	0	0	-121	1.74	B11
Interior	A5/5	WM	90.4	Gravity	0.4	0.0	0	0	0	0		
mullion		2		H. Seismic	-7.7	0.0	0	-2	0	0		
(S3x7.5)				0.314 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	-7.5	0.0	0	-2	0	0	0.41	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-5.8	0.3	-0.5	-9	-10	7		
mullion at		9		H. Seismic	-8.4	9.5	-9.2	-98	-227	254		
the roof				0.314 Gravity	-1.8	0.1	-0.1	-3	-3	2		
(additive)		n=810		Total	-16.1	9.8	-9.8	-110	-240	263	0.66	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	1.7	0.7	0.2	0	0	-29		
at top of		809		H. Seismic	-12.4	5.1	2.9	1	0	-211		
window				0.314 Gravity	-0.5	0.2	0.1	0	0	-9		
(buckling)		n=810		Total	-11.2	6.0	3.2	1	0	-249	1.66	B14

Table 23. Forces, Moments, and DCRs (L8, 30-ft tower, all windows, $\theta_L = 90°$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.3	Gravity	-74.8	-0.2	0.1	1	2	139		
at Base		33		H. Seismic	27.9	-44.1	7.1	615	1693	9359		
				0.314 Gravity	23.5	-0.1	0.0	0	1	44		
(counteract)		n=119		Total	-23.5	-44.4	7.2	617	1696	9542	0.46	B1
Bent Column	S1	NPCM	160.2	Gravity	-27.1	9.2	0.0	-2	-16	-787		
at Shaft		42		H. Seismic	28.2	38.0	6.3	-318	-224	-1385		
				0.314 Gravity	8.5	2.9	0.0	-1	-5	-247		
(counteract)		n=536		Total	9.7	50.1	6.3	-321	-244	-2420	0.43	B2
Bent Beam	S1	TBNT	92.9	Gravity	-6.3	-7.3	0.2	0	1	-513		
at Shaft		532		H. Seismic	33.5	-9.1	1.7	289	359	-1508		
				0.314 Gravity	2.0	-2.3	0.0	0	0	-161		
(counteract)		n=508		Total	29.2	-18.7	1.9	290	360	-2183	0.66	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-7.5	-1.2	0.0	0	0	0		
at center of		538		H. Seismic	-6.4	-9.4	-3.3	290	0	0		
tower roof				0.314 Gravity	-2.4	-0.4	0.0	0	0	0		
(additive)		n=531		Total	-16.3	-10.9	-3.3	291	0	0	0.23	B4
Bent/roof	S2	EB	-	Gravity	-3.3	14.6	-0.3	2	0	0		
beam conn	S4/13,19	509		H. Seismic	-12.2	23.1	-3.0	5	0	0		
16WF88	S5/17,18			0.314 Gravity	-1.0	4.6	-0.1	1	0	0		
(additive)		n=514		Total	-16.4	42.4	-3.3	7	0	0	0.49	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-8.0	0.8	-2.6	-2	4	-99		
mullions at		3		H. Seismic	-21.9	15.8	-9.2	-29	183	-652		
their base				0.314 Gravity	-2.5	0.3	-0.8	-1	1	-31		
(additive)	col. 1-4	n=603		Total	-32.4	16.9	-12.6	-31	189	-783	1.09	B6
Corner	S4/A,2,5	CLCM	-	Gravity	-15.9	1.8	0.1	1	0	147		
mullion/		6		H. Seismic	27.1	16.5	4.1	20	18	565		
10WF conn				0.314 Gravity	4.8	0.5	0.0	0	0	44		
(counteract)	col. 1-4	n=533		Total	16.0	18.8	4.2	21	18	756	7.86	B7
Corner	S4/8,9	CLCM	-	Gravity	-8.2	0.8	-2.6	-2	-95	-68		
mullion/		3		H. Seismic	21.9	15.8	-9.2	-29	-241	-171		
10WF conn				0.314 Gravity	2.6	0.3	-0.8	-1	-30	-21		
(counteract)	col. 5	n=506		Total	16.3	16.9	-12.6	-31	-366	-260	1.76	B8
10WF72	S2,S3	RAD	43.4	Gravity	-1.8	16.2	0.1	0	-3	-550		
at 16WF88	S4/A	525		H. Seismic	-16.5	27.3	4.1	18	-197	-1576		
shaft roof				0.314 Gravity	-0.6	5.1	0.0	0	-1	-173		
(additive)		n=515		Total	-18.9	48.6	4.2	18	-201	-2298	0.90	B9
Cab floor	S2	CEPG	63.0	Gravity	2.5	-2.7	0.2	0	-11	191		
12WF27		602		H. Seismic	9.6	-4.6	0.6	0	-35	288		
near Col 5				0.314 Gravity	0.8	-0.9	0.1	0	-3	60		
(additive)		n=604		Total	12.9	-8.2	0.8	0	-49	539	0.77	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-0.8	0.0	-0.2	0	0	-1		
at base of		703		H. Seismic	-14.6	1.8	-12.0	0	0	-87		
window				0.314 Gravity	-0.2	0.0	0.0	0	0	0		
(additive)		n=703		Total	-15.6	1.8	-12.1	0	0	-88	1.88	B11
Interior	A5/5	WM	90.4	Gravity	0.5	0.0	0	0	0	0		
mullion		2		H. Seismic	-10.6	0.0	0	0	0	0		
(S3x7.5)				0.314 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	-10.3	0.0	0	1	0	-1	0.58	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-2.6	0.3	0.2	5	4	9		
mullion at		9		H. Seismic	-9.5	1.9	5.1	50	147	51		
the roof				0.314 Gravity	-0.8	0.1	0.1	2	1	3		
(additive)		n=810		Total	-12.9	2.3	5.3	57	152	63	0.33	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.8	0.7	0.2	0	0	-26		
at top of		809		H. Seismic	-7.9	6.2	3.7	0	0	-257		
window				0.314 Gravity	-0.3	0.2	0.0	0	0	-8		
(buckling)		n=810		Total	-7.4	7.1	3.9	-1	0	-292	1.85	B14

Table 24. Corner Mullion Base Connection Evaluation - Forces, Moments, and DCRs (L6, 30-ft tower, no windows, θ_L = 90°, S_{DS} = 1.07g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Corner	S-4/A,2,5	CLCM	-	Gravity	-10.5	-1.2	1.6	3	83	-58		
mullion/	S-4/8,9	1		H. Seismic	6.1	-14.5	14.1	131	374	-267		
10WF conn				0.314 Gravity	3.3	-0.4	0.5	1	26	-18		
		n=503		Total	-1.1	-16.1	16.3	134	483	-343	2.33	B7
Corner	S-4/A,2,5	CLCM	-	Gravity	-9.0	-0.2	-1.6	0	-73	-49		
mullion/	S-4/8,9	3		H. Seismic	6.1	-8.2	-8.3	-142	-314	-230		
10WF conn		12" COL		0.314 Gravity	2.8	-0.1	-0.5	0	-23	-16		
		n=506		Total	-0.1	-8.5	-10.4	-142	-409	-295	2.04	B8
Corner	S-4/A,2,5	CLCM	-	Gravity	-13.9	2.2	-0.1	0	9	68		
mullion/	S-4/8,9	4		H. Seismic	8.8	26.1	-3.9	146	69	1623		
10WF conn				0.314 Gravity	4.4	0.7	0.0	0	3	21		
		n=534		Total	-0.8	29.0	-4.0	146	81	1711	11.67	B7
Corner	S-4/A,2,5	CLCM	-	Gravity	-12.5	1.5	0.0	3	-11	84		
mullion/	S-4/8,9	5		H. Seismic	7.9	24.3	-3.8	148	-83	1373		
10WF conn				0.314 Gravity	3.9	0.5	0.0	1	-3	26		
		n=532		Total	-0.7	26.3	-3.8	153	-98	1483	10.13	B7
Corner	S-4/A,2,5	CLCM	-	Gravity	-15.1	-0.5	0.1	3	-1	-31		
mullion/	S-4/8,9	6		H. Seismic	12.0	-28.5	2.6	129	-56	-1954		
10WF conn				0.314 Gravity	4.7	-0.2	0.0	1	0	-10		
		n=533		Total	1.6	-29.2	2.7	133	-57	-1995	14.31	B7
										Average DCR =	8.09	

Table 25. Corner Mullion Base Connection Evaluation - Forces, Moments, and DCRs (Seismic Factor, F_1 = 0.1055, L6, 30-ft tower, no windows, θ_L = 90°, S_{DS} = 0.113g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Corner	S-4/A,2,5	CLCM	-	Gravity	-10.5	-1.2	1.6	3	83	-58		
mullion/	S-4/8,9	1		H. Seismic	0.6	-1.5	1.5	13.8	39.4	-28.1		
10WF conn				0.314 Gravity	0.3	0.0	0.0	0.1	2.0	-1.4		
		n=503		Total	-9.6	-2.8	3.1	16	124	-87	0.55	B7
Corner	S-4/A,2,5	CLCM	-	Gravity	-9.0	-0.2	-1.6	0	-73	-49		
mullion/	S-4/8,9	3		H. Seismic	0.6	-0.9	-0.9	-15.0	-33.1	-24.3		
10WF conn		12" COL		0.314 Gravity	0.2	0.0	0.0	0.0	-1.8	-1.2		
		n=506		Total	-8.2	-1.1	-2.5	-15	-107	-75	0.48	B8
Corner	S-4/A,2,5	CLCM	-	Gravity	-13.9	2.2	-0.1	0	9	68		
mullion/	S-4/8,9	4		H. Seismic	0.9	2.8	-0.4	15.4	7.3	171.1		
10WF conn				0.314 Gravity	0.3	0.1	0.0	0.0	0.2	1.6		
		n=534		Total	-12.7	5.0	-0.5	16	17	240	1.32	B7
Corner	S-4/A,2,5	CLCM	-	Gravity	-12.5	1.5	0.0	3	-11	84		
mullion/	S-4/8,9	5		H. Seismic	0.8	2.6	-0.4	15.6	-8.8	144.8		
10WF conn				0.314 Gravity	0.3	0.0	0.0	0.1	-0.3	2.1		
		n=532		Total	-11.4	4.1	-0.4	19	-20	231	1.29	B7
Corner	S-4/A,2,5	CLCM	-	Gravity	-15.1	-0.5	0.1	3	-1	-31		
mullion/	S-4/8,9	6		H. Seismic	1.3	-3.0	0.3	13.6	-5.9	-206.1		
10WF conn				0.314 Gravity	0.4	0.0	0.0	0.1	0.0	-0.8		
		n=533		Total	-13.5	-3.5	0.4	17	-7	-238	1.29	B7
										Average DCR =	0.99	

5 Type L Seismic Upgrade Development

Description of Proposed Upgrade for All Type L Towers

The upgrade approach proposed here consists of welding very deep structural tube (ST 20 x 4 x ½) members to the base of each mullion in a pentagon configuration (see Figure 11). These members must also be anchored to the horizontal portion of the concrete bents. This connection only needs to transfer shear forces, not moments. The tubes must be welded to the mullions and each other to form a continuous pentagon. This pentagon structure will form a very stiff foundation to greatly reduce rotations by itself. The tubes will also relieve the overstressed mullion connections to the shaft roof beams. These tubes will never allow enough rotation to significantly stress the existing mullion base plates and bolts. The tubes by themselves, however, will not bring the story drift down to acceptable levels. Figure 11 also shows the stiffening and strengthening of the mullions, by welding 5 in. x 1.5 in. plates on both faces of the mullions. This, together with the ST 20 x 4 x ½ tubes around the perimeter, significantly reduces cab deflections and DCRs in the mullions.

Salinas Tower Upgrade – Shaft Members and Connections

Analysis Case L13* was used to evaluate the application of the seismic upgrade described above to the Type L tower shaft at the 50-ft tall Salinas tower. Because the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90°$) gave one of the worst-case loading for critical shaft elements (Case L1), the same seismic loading is used to evaluate the upgraded tower in Case L13.

* The San Carlos tower upgrade was first evaluated with the L12 analysis case because the upgrade was developed for the more vulnerable cab of the San Carlos tower.

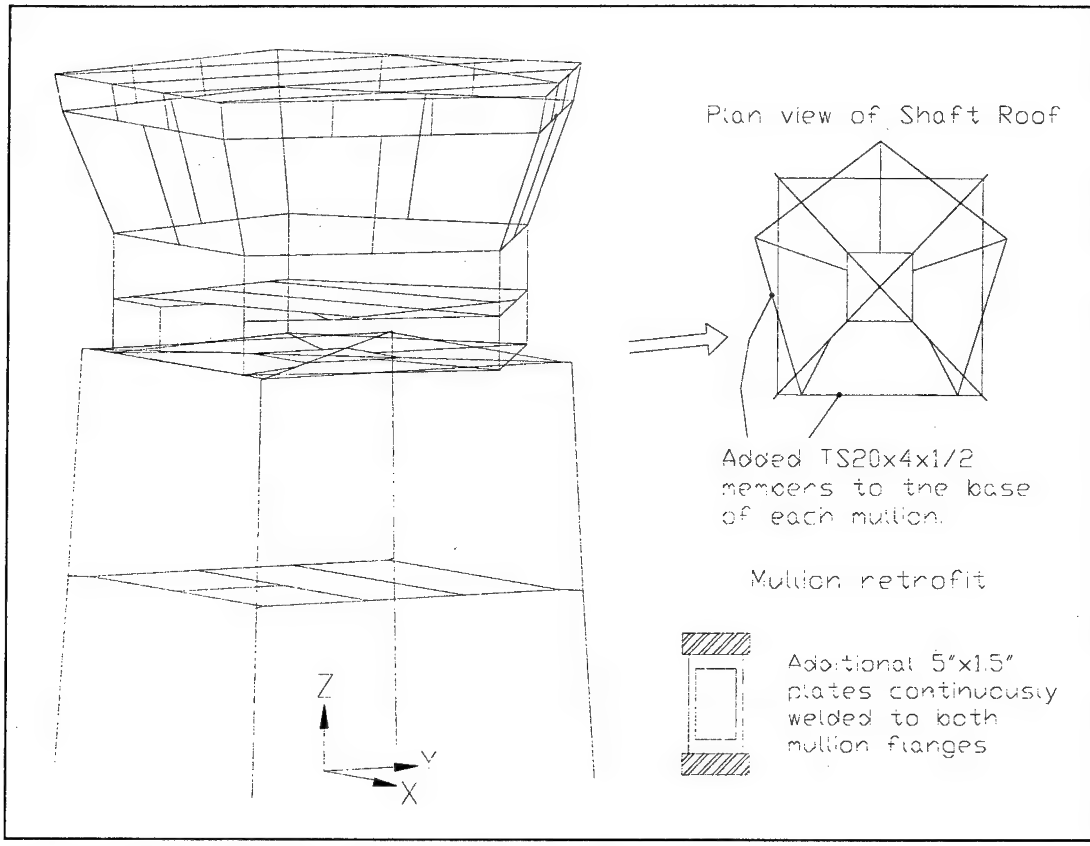


Figure 11. Finite element mesh for the upgraded 30-ft tall San Carlos tower.

Modal Analysis Results and Deflections

Tables 26, 27, and 28 summarize the 50-ft Salinas tower upgrade modal analysis results, deflections, and story drifts. Drifts fall well within allowable values.

Shaft Member and Connection Evaluation

Table 29 presents the force and moment summary for the most critically stressed members and connections. The column bent is slightly overstressed with a DCR of 2.21, but this is based on conservative modeling that will cause the bent to be more heavily loaded than reality. The DCRs for all other members and connections in the shaft fall below 2.0, demonstrating that the proposed upgrade is adequate for the tower shaft.

Table 26. Salinas (Type L – 50 ft) Shaft Upgrade Modal Analysis Results.

Mode #	L13 Period (sec)	Cumulative Participating Mass (%) X-dir	Y-dir	Z-dir	Mode of Vibration
1	0.692	2.9	0.5	0.0	1st Torsion
2	0.562	3.6	63.3	0.0	1st Y-Lateral
3	0.542	65.8	63.8	0.0	1st X-Lateral
4	0.249	65.8	68.1	1.2	2nd Y-Lateral
5	0.227	66.9	68.2	1.2	2nd Torsion
6	0.184	68.4	68.2	1.2	2nd X-Lateral
7	0.147	68.4	68.9	10.5	1st Vertical/cab floor & roof
8	0.142	69.1	68.9	10.5	3rd Torsion
9	0.129	69.2	69.6	11.3	2nd Vertical/cab roof & floor
10	0.114	82.3	69.6	11.4	3rd X-Lateral/cab rocking
11	0.113	82.4	83.2	11.4	3rd Y-Lateral/cab rocking
15	0.097	84.0	84.3	19.5	3rd Vertical/4th floor & roof
18	0.094	84.5	84.4	30.6	4th Vertical/2nd & 3rd floor
28	0.077	85.1	85.3	38.2	4th Torsion
48	0.051	86.1	89.6	47.7	4th Y-Lateral
49	0.049	87.7	89.6	47.9	4th X-Lateral
50	0.048	89.6	90.9	48.0	4th X-Lateral
51	0.047	90.9	90.9	48.0	4th X-Lateral

Table 27. Salinas Shaft Upgrade Selected Horizontal SRSS Deflections.

Location	L13 Joint #	δ_{xe} (in.)
2nd Floor, δ_1	204	0.27
3rd Floor, δ_2	304	1.11
4th Floor, δ_3	404	2.41
Shaft Roof, δ_4	508	3.92
Cab Floor, δ_5	607	4.23
Bottom of Windows	706	4.63
Top of Windows	810	5.11
Cab Roof, δ_6	911	5.28

Table 28. Salinas Tower Upgrade Story Drifts.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Analysis Case	Joint #	Elastic Defl δ_x (in.)	Calc Story Drift Δ_c (in.)
Ground Floor	0			L13	102	0.00	
2nd Floor	134	134	2.68	L13	204	0.27	0.27
3rd Floor	278	144	2.88	L13	304	1.11	0.84
4th Floor	422	144	2.88	L13	404	2.41	1.30
Shaft Roof	582	160	3.2	L13	508	3.92	1.51
Shaft Roof	582	160	3.2	L13	513	3.87	
Cab Floor	620	38	0.76	L13	607	4.23	0.37
Cab Roof	777	157	3.14				

Table 29. Salinas Upgrade Forces, Moments, and DCRs (L13, 50-ft tower, all windows, $\theta_L = 90°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.1	Gravity	-113.7	1.3	0.0	0	-5	202		
at Base		12		H. Seismic	78.6	117.4	-7.7	-574	-3227	47246		
				0.300 Gravity	34.1	0.4	0.0	0	-1	61		
(counteract)		n=102		Total	-1.0	119.1	-7.7	-574	-3233	47508	1.10	B1
Bent Column	S1	NPCM	160.1	Gravity	-29.4	12.8	-0.3	2	16	-1315		
at Shaft		42		H. Seismic	78.1	75.7	-2.8	349	343	-8456		
				0.300 Gravity	8.8	3.8	-0.1	1	5	-395		
(counteract)		n=536		Total	57.5	92.4	-3.2	351	364	-10166	1.30	B2
Bent Beam	S1	TBNT	92.9	Gravity	-8.8	-22.5	-0.2	2	4	-971		
at Shaft		553		H. Seismic	44.1	-47.4	-13.6	347	316	-8138		
				0.300 Gravity	2.6	-6.8	-0.1	1	1	-291		
(counteract)		n=508		Total	38.0	-76.7	-13.9	349	321	-9401	2.21	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-8.5	-0.4	0.0	1	0	0		
at center of		538		H. Seismic	-8.4	-50.3	-3.4	276	0	0		
tower roof				0.300 Gravity	-2.6	-0.1	0.0	0	0	0		
(additive)		n=531		Total	-19.5	-50.8	-3.4	277	0	0	0.53	B4
Bent/roof	S2	EB	-	Gravity	-3.7	0.6	0.0	0	0	0		
beam conn	S4/13,19	505		H. Seismic	-33.2	42.2	1.2	1	0	0		
16WF88	S5/17,18			0.300 Gravity	-1.1	0.2	0.0	0	0	0		
(additive)		n=507		Total	-37.9	43.0	1.2	1	0	0	0.50	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-14.2	12.5	6.5	0	208	453		
mullions at		4		H. Seismic	-7.9	9.5	37.5	-15	1251	281		
their base				0.300 Gravity	-4.3	3.8	2.0	0	62	136		
(additive)	col. 1-4	n=534		Total	-26.3	25.8	46.0	-15	1521	870	1.51	B6a
Corner	S4/A,2,5	CLCM	-	Gravity	-14.2	12.5	6.5	0	0	0		
mullion/		4		H. Seismic	7.9	9.5	37.5	-15	0	0		
10WF conn				0.300 Gravity	4.3	3.8	2.0	0	0	0		
(counteract)	col. 1-4	n=534		Total	-2.1	25.8	46.0	-15	0	0	0.83	B7
Corner	S4/8,9	CLCM	-	Gravity	-9.4	4.9	-4.4	-1	0	0		
mullion/		3		H. Seismic	21.8	16.6	-7.3	-52	0	0		
10WF conn				0.300 Gravity	2.8	1.5	-1.3	0	0	0		
(counteract)	col. 5	n=506		Total	15.2	23.0	-13.0	-53	0	0	0.31	B8
10WF72	S2,S3	RAD	43.4	Gravity	-0.2	9.9	0.0	0	0	-261		
at 16WF88	S4/A	525		H. Seismic	-3.8	17.8	2.5	0	74	-898		
shaft roof				0.300 Gravity	-0.1	3.0	0.0	0	0	-78		
(additive)		n=515		Total	-4.1	30.7	2.5	0	74	-1237	0.46	B9
Cab floor	S2	CEPG	63.0	Gravity	5.8	-2.7	0.0	0	0	191		
12WF27		602		H. Seismic	11.7	-2.1	-0.2	0	15	129		
near Col 5				0.300 Gravity	1.7	-0.8	0.0	0	0	57		
(additive)		n=604		Total	19.3	-5.6	-0.2	0	15	378	0.49	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	-0.9	0.2	0.0	0	0	-7		
at base of		705		H. Seismic	-6.4	1.6	0.7	0	0	-79		
window				0.300 Gravity	-0.3	0.1	0.0	0	0	-2		
(additive)		n=706		Total	-7.6	1.9	0.6	0	0	-88	1.23	B11
Interior	A5/5	WM	90.4	Gravity	0.2	0.0	0	0	0	0		
mullion		2		H. Seismic	-11.5	0.0	0	0	0	0		
(S3x7.5)				0.300 Gravity	0.0	0.0	0	0	0	0		
(buckling)		n=807		Total	-11.4	0.0	0	0	0	0	0.63	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-2.6	-0.2	0.5	6	11	-4		
mullion at		6		H. Seismic	-1.6	-6.6	6.0	48	172	-175		
the roof				0.300 Gravity	-0.8	-0.1	0.1	2	3	-1		
(additive)		n=801		Total	-4.9	-6.8	6.6	56	186	-180	0.24	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	0.5	0.5	0.1	0	0	-20		
at top of		809		H. Seismic	-8.7	6.3	3.9	0	0	-262		
window				0.300 Gravity	0.1	0.2	0.0	0	0	-6		
(buckling)		n=810		Total	-8.1	7.0	4.0	0	0	-288	1.86	B14
TS 20x4x1/2	NA	TUBE	70.7	Gravity	-14.5	-7.3	0.0	30	0	534		
around mull.		557		H. Seismic	-15.7	-13.3	0.4	63	28	940		
perimeter	retrofit			0.314 Gravity	-4.6	-2.3	0.0	9	0	168		
(additive)	member	n=534		Total	-34.7	-22.9	0.4	102	28	1641	0.41	B15

San Carlos Tower Upgrade – Cab Members and Connections

Analysis Case L12 was used to evaluate the application of the seismic upgrade to the most critical Type L tower cab at the 30-ft tall San Carlos tower. Because the full seismic spectrum at 90 degrees to the X-axis ($\theta_L = 90°$) gave the worst case loading for critical cab elements (Case L6), the same seismic loading was used to evaluate the upgraded tower in Case L12.

Modal Analysis Results and Deflections

Tables 30, 31, and 32 summarize the 30-ft San Carlos tower upgrade modal analysis results, deflections, and story drifts. Calculated story drifts exceed allowable values by 50 percent. In an actual earthquake, these deflections would be reduced by the additional stiffening of the mullions by the deep structural tubing member that prevents deflections and rotations of the mullions up to the depth of the structural tubing (20 in.). Also, for a moment frame structure (for the upgraded configuration), FEMA 302 gives an R-value of 8 and a deflection amplification value, C_d, of 5.5. For design purposes, the effective story drift could be reduced to acceptable levels by dividing by R (8) and multiplying by C_d (5.5).

The addition of these tubes reduces rotations at the shaft roof mullion connections by more than an order of magnitude. Table 33 presents the rotations at the base of these mullions before (Case L6) and after the upgrade (Case L12). The difference in elevation between the cab roof and shaft roof in this model is 195 in. (777 – 582 in.). The rotations at the mullion connections to the shaft roof in Table 33 were multiplied by the 195-in. mullion height to give the deflections due to rotations alone. The SRSS deflections, δ for both the x and y rotations (R_x and R_y respectively), were calculated as follows:

$$\delta = 195\,inches\sqrt{R_x^2 + R_y^2} \qquad \text{[Eq 26]}$$

These deflections are given in Table 33. This shows the influence of the rotations on deflections and the effectiveness of the proposed upgrade in reducing these deflections. The story drifts in Table 32 are greater than these values due to bending of the upgraded mullions.

Cab Member and Connection Evaluation

Table 34 summarizes the force and moment for the most critically stressed members and connections. The greatest DCR was 1.56 (below 2.0), demonstrating the effectiveness of the proposed upgrade. Table 34 shows that the upgraded

mullion has a DCR of only 1.47. The capacity of this member is defined in Appendix B6a and the applied loads in this appendix are those shown in Table 34 for the L12 analysis case. The bottom of Table 34 shows that the TS 20 x 4 x ½ structural tube used to upgrade this tower has a DCR of only 0.61, demonstrating its effective performance. The capacity of this member is defined in Appendix B15.

Table 30. San Carlos (Type L – 30 ft) Cab Upgrade Modal Analysis Results.

Mode #	L12				Mode of Vibration
	Period (sec)	Cumulative Participating Mass (%)			
		X-dir	Y-dir	Z-dir	
1	0.709	3.1	0.4	0.0	1st Torsion/cab
2	0.559	4.8	13.8	0.1	1st Y-Lateral/cab
3	0.539	16.5	17.0	0.1	1st X-Lateral/cab
6	0.270	17.0	17.1	0.1	2nd Torsion
9	0.183	17.0	27.8	9.7	1st Vertical & Y-Lateral
10	0.179	17.0	28.7	12.2	2nd Vertical/cab roof
13	0.166	73.1	28.8	12.3	2nd X-Lateral
14	0.163	73.2	74.2	13.9	2nd Y-Lateral
15	0.143	73.5	74.2	17.0	Vertical/X-cab rocking
26	0.097	75.5	75.3	30.0	3rd Vertical/2nd floor
33	0.083	75.5	75.7	43.0	4th Vertical/shaft roof
47	0.061	75.9	75.8	46.0	3rd Torsion
60	0.036	85.6	95.1	51.1	4th Y-Lateral
61	0.034	98.7	97.5	51.1	4th X-Lateral

Table 31. San Carlos Cab Upgrade Selected Horizontal SRSS Deflections.

Location	L12	
	Joint #	δ_{xe} (in.)
2nd Floor, δ_1	204	0.08
Shaft Roof, δ_2	508	0.33
Cab Floor, δ_3	607	0.66
Bottom of Windows	703	1.61
Top of Windows	805	4.52
Cab Roof, δ_4	907	5.40

Table 32. San Carlos Tower Upgrade Story Drifts.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Analysis Case	Joint #	Elastic Defl δ_x (in.)	Calc Story Drift Δ_c (in.)
Ground Floor	0			L12			
2nd Floor	134	134	2.68	L12	204	0.08	0.08
Shaft Roof	294	160	3.2	L12	508	0.33	0.25
Cab Floor	332	38	0.76	L12	603	0.66	0.32
Cab Roof	489	157	3.14	L12	907	5.404	4.75

Table 33. Rotations at the Base of the Mullions Before and After the Upgrade.

Mullion (Col #*)	Joint Number	Case L6 Rotations and Deflections			Case L12 Rotations and Deflections		
		R_x (radians)	R_y (radians)	δ (in.)	R_x (radians)	R_y (radians)	δ (in.)
1	503	0.0451	0.0013	8.80	0.0049	0.0006	0.96
2	534	0.0506	0.0221	10.77	0.0009	0.0035	0.70
3	533	0.0184	0.0607	12.37	0.0064	0.0006	1.25
4	532	0.0613	0.0245	12.87	0.0013	0.0044	0.89
5	506	0.0546	0.0008	10.65	0.0052	0.0009	1.03

* Drawing S3 (Corlett & Spackman 1966).

Table 34. San Carlos Upgrade Forces, Moments, and DCRs (L12, 30-ft tower, no windows, $\theta_L = 90°$, $S_{DS} = 1.07g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bent Column	S1	NPCM	134.3	Gravity	-67.6	-5.2	0.6	4	50	-62		
at Base		32		H. Seismic	16.8	-54.1	5.0	314	1222	-1292		
				0.314 Gravity	21.2	-1.6	0.2	1	16	-19		
(counteract)		n=118		Total	-29.6	-60.9	5.7	319	1288	-1373	0.49	B1
Bent Column	S1	NPCM	160.2	Gravity	-30.0	14.7	-0.4	2	18	-1318		
at Shaft		42		H. Seismic	16.0	41.5	-4.3	195	294	-1335		
				0.314 Gravity	9.4	4.6	-0.1	1	6	-414		
(counteract)		n=536		Total	-4.6	60.8	-4.9	198	318	-3067	0.51	B2
Bent Beam	S1	TBNT	92.9	Gravity	-10.3	-24.1	-0.2	0	4	-983		
at Shaft		553		H. Seismic	22.5	-73.1	-8.2	-164	184	-1504		
				0.314 Gravity	3.2	-7.6	-0.1	0	1	-309		
(counteract)		n=508		Total	15.4	-104.7	-8.5	-164	190	-2796	1.56	B3
Bent conn	S1/2,3	TBNT	-	Gravity	-7.0	0.6	0.1	-3	0	0		
at center of		531		H. Seismic	-14.6	11.4	2.7	-134	0	0		
tower roof				0.314 Gravity	-2.2	0.2	0.0	-1	0	0		
(additive)		n=531		Total	-23.7	12.1	2.8	-138	0	0	0.33	B4
Bent/roof	S2	EB	-	Gravity	-4.2	9.7	0.0	1	0	0		
beam conn	S4/13,19	509		H. Seismic	-20.2	9.9	0.2	2	0	0		
16WF88	S5/17,18			0.314 Gravity	-1.3	3.0	0.0	0	0	0		
(additive)		n=514		Total	-25.6	22.7	0.1	3	0	0	0.26	B5
Corner	S4/A,2,3	CLCM	37.9	Gravity	-11.4	3.9	4.4	0	143	198		
mullions at		1		H. Seismic	-6.6	22.3	12.7	20	616	2461		
their base				0.314 Gravity	-3.6	1.2	1.4	0	45	62		
(additive)	col. 1-4	n=503		Total	-21.6	27.4	18.5	20	805	2722	1.47	B6a
Corner	S4/A,2,5	CLCM	-	Gravity	-11.4	3.9	4.4	0	0	0		
mullion/		1		H. Seismic	6.6	22.3	12.7	20	0	0		
10WF conn				0.314 Gravity	3.6	1.2	1.4	0	0	0		
(counteract)	col. 1-4	n=503		Total	-1.2	27.4	18.5	20	0	0	0.52	B7
Corner	S4/8,9	CLCM	-	Gravity	-10.0	5.2	-4.1	0	0	0		
mullion/		3		H. Seismic	6.4	13.4	-9.8	104	0	0		
10WF conn				0.314 Gravity	3.1	1.6	-1.3	0	0	0		
(counteract)	col. 5	n=506		Total	-0.4	20.2	-15.1	105	0	0	0.29	B8
10WF72	S2,S3	RAD	43.4	Gravity	-0.2	9.4	0.0	0	0	155		
at 16WF88	S4/A	525		H. Seismic	-3.3	11.3	1.2	-1	12	426		
shaft roof				0.314 Gravity	-0.1	2.9	0.0	0	0	49		
(additive)		n=515		Total	-3.5	23.6	1.2	-1	12	630	0.22	B9
Cab floor	S2	CEPG	63.0	Gravity	5.6	-2.7	0.0	0	0	191		
12WF27		602		H. Seismic	8.1	-1.4	0.2	0	11	89		
near Col 5				0.314 Gravity	1.8	-0.9	0.0	0	0	60		
(additive)		n=604		Total	15.5	-5.0	0.2	0	12	340	0.43	B10
C 4 x 7.25	S3	CEPG	192.0	Gravity	0.0	0.0	-0.1	0	0	-4		
at base of		701		H. Seismic	-4.5	0.7	0.0	0	0	-68		
window				0.314 Gravity	0.0	0.0	0.0	0	0	-1		
(additive)		n=702		Total	-4.5	0.7	-0.1	0	0	-74	0.91	B11
Interior	A5/5	WM	90.4	Gravity	0.3	0.0	0	0	0	0		
mullion		2		H. Seismic	-0.1	0.0	0	0	0	-1		
(S3x7.5)				0.314 Gravity	-0.1	0.0	0	0	0	0		
(buckling)		n=807		Total	0.1	0.1	0	0	0	-2	0.03	B12
Corner	S4/1,2,3,7	CUCM	26.7	Gravity	-5.2	0.8	0.0	0	1	21		
mullion at		7		H. Seismic	-4.3	3.0	2.6	-26	71	79		
the roof				0.314 Gravity	-1.6	0.2	0.0	0	0	7		
(additive)		n=805		Total	-11.1	4.0	2.6	-27	71	108	0.11	B13
C 6 x 8.2	S3	CEPG	237.0	Gravity	-0.1	0.0	0.0	0	0	4		
at top of		808		H. Seismic	-9.4	1.2	0.2	0	0	95		
window				0.314 Gravity	0.0	0.0	0.0	0	0	1		
(buckling)		n=808		Total	-9.5	1.2	0.2	0	0	101	0.76	B14
TS 20x4x1/2	NA	TUBE	23.8	Gravity	-5.0	13.8	0.0	21	0	329		
around mull.		552		H. Seismic	-8.0	83.6	2.8	97	68	1990		
perimeter	retrofit			0.314 Gravity	-1.6	4.3	0.0	7	0	103		
(additive)	member	n=506		Total	-14.6	101.7	2.8	125	68	2423	0.61	B15

6 San Luis Obispo Analysis Results

As with the Type L towers, the San Luis Obispo tower was analyzed using SAP2000 and MathCAD member and connection evaluation files. The first section of these results focuses on the tower shaft evaluation, which was controlled by the conservative assumption that all the cab windows remain intact and perform as fully effective shear walls (Cases SLO1, SLO2, SLO3, and SLO3a). The second section focuses on the cab evaluation, which was a more realistic model that does not include any stiffening from the cab windows. Table 7 summarized all the cases used to evaluate the San Luis Obispo tower.

Each section in this report presents the modal analysis results, deflections, story drifts, forces, and moments of critical members and connections and the resulting DCRs. The towers may fail evaluation based on either exceeding drift limits, or high DCRs that lead to collapse of the towers. Code-based design resistance factors have been dropped (i.e., set equal to 1.0) for the purpose of evaluating structural members. The design resistance factors were included in the evaluation of connections, as such failure must be prevented, because they would fail in a more brittle manner than the structural members would. Appendix D1 through D12 shows the MathCAD models used to evaluate each member and connection, and these appendix numbers are given in each force, moment, and DCR summary table in this chapter.

Shaft Members and Connections

The most critical condition for the shaft evaluation is the extreme condition where all cab windows remain intact and act as fully effective shear walls (SLO1 – SLO3a). This decreases the fundamental period of the structure and increases the effective acceleration, and therefore loads the shaft members and connections more severely. As with the Type L evaluation, this assumption provides an upper bound basis for evaluating the effect of the windows remaining intact. This effect was modeled by defining the shear stiffness of the windows if they act as a shear block. For the San Luis Obispo cab, the connections between the mullions and structural tubing above and below the windows are fixed, so no modification of these connections is needed to model the windows acting as shear walls (such a modification was needed for the Type L evaluation). Analysis

Cases SLO1, SLO2, and SLO3 evaluate the performance of the San Luis Obispo tower with the windows acting as shear walls with the full seismic response spectrum acting at 90, 45, and 0 degrees to the X-axis (θ_L = 90°, 45°, and 0°), respectively. The X-axis orientation is shown on the model plot in Figure 7. This axis is along the front face of the tower as shown on Drawing S-2 (Leo A Daly 1981) of the San Luis Obispo tower.

Modal Analysis Results and Deflections

Table 35 presents the primary modes of vibration for the San Luis Obispo tower, Cases SLO1, SLO2, and SLO3 evaluations. The cumulative participating mass shows that a greater portion of the mass participates in the first X and Y lateral modes than in the cases when the cab windows are not acting as shear walls (SLO5, SLO6, and SLO7 in Table 44). This demonstrates that the cab is more coupled with the tower shaft vibration in the first mode, and this will more heavily load the shaft components due to higher effective accelerations.

Table 36 summarizes maximum lateral deflections at each floor level and other key locations in the control tower for the shaft evaluation cases. All deflections are the SRSS of the total X and Y lateral deflections. The shaft deflections are greater for the shear wall window cases (SLO1 – SLO3) because of the coupling and higher effective accelerations described above. The greatest deflection at the top of the shaft is 1.32 in. for case SLO3 (θ_L = 0°). These deflections are well within story drift limits. Therefore, story drifts are not considered further in this section, but will be examined in the next section for the cab. Cab deflections for the cases of shear wall windows are unrealistically low because of the shear wall window stiffening. These cases are intended for shaft evaluation only, and will produce unrealistic results in the cab.

Shaft Member and Connection Evaluation

Tables 37 through 39 give the force and moment summary for the most critically stressed members and connections. These tables also present the DCRs for each of these critical components. Tables 37 through 43 and 47 through 51 presents the same information for the San Luis Obispo tower evaluation as described in the Type L tower evaluation. For this tower, component capacities are defined in Appendices D1 through D12, using the forces and moments calculated from analysis case SLO1.

Case SLO3, with the full seismic spectrum at 0 degrees to the X-axis (θ_L = 0°), is the most critical loading condition for shaft members and connections. The highest DCR is in the braces. Table 39 shows that a brace in the plane of the X-

axis (BR107), located at the first floor of the building, has a DCR of 3.06. Other braces (such as BR508) shown in Table 39 have DCRs over 2.0. These high DCRs indicate that several braces will buckle. Tables 40 and 41 give the same information for all the braces in the plane of the X-axis. DCRs are much smaller for the braces out-of-plane to the X-axis, as the building is not significantly seismically loaded in this direction. Table 40 reveals that a brace at the third floor (BR407) has the highest DCR, with a value of 4.25. Table 40 also shows that all the braces up to the junction level (fourth floor) will buckle, as indicated by DCRs greater than 2.0.

The impact of buckling braces was studied further by removing all the braces on one side of the building, in the plane of the X-axis. In an earthquake, all the braces acting in compression on one side of the building could buckle, which would increase the load of the braces in tension on the other side. When the building cycles back in the other direction, the braces that had been in tension will go into compression and will also buckle, while the braces that had buckled on the other side will effectively carry load in tension. Therefore, if the braces in tension have adequate capacity alone for the full lateral load, the building will remain stable and prevent collapse. Many buildings are designed with tension only braces, where the design assumption is that the slender tension braces will buckle. The braces in the San Luis Obispo tower could have been designed as tension only, but this remains unclear. These braces are L6 x 6 x ½ with a kl/r value of only 124, which is not that slender.

The San Luis Obispo tower was evaluated with tension only braces in analysis Case SLO3a. The loading was identical to Case SLO3. Table 42 summarizes the forces, moments, and DCRs for all the tension braces in the plane of the X-axis. These braces are now loaded with greater axial tension force than when the compression braces were in place, and the DCRs are still much lower than the values of the compression braces in Case SLO3. The DCRs shown in Table 42 include the moments on these eccentrically loaded braces. Brace 202, between floors 2 and 3 has a high DCR, with a value of 2.09. This indicates that this brace will begin to yield in tension on one edge of the angle. However, the tower will remain very stable, because collapse would require yielding of the entire gross area of the section. This same brace was evaluated for gross section yielding with the same loading, which gives a DCR of only 0.53 (see Table 42). Therefore, the tower would remain stable, with only slight yielding of the braces in combined tension and moment. Deflections would be somewhat greater for the case with tension only braces than those shown in Table 36, for Case SLO3. However, they will remain far below acceptable story drift limits.

All the other components were evaluated with the tension only analysis case (SLO3a), and a summary of this evaluation is shown in Table 43. Comparing Table 43 with Table 39 (Case SLO3) shows that the loading and DCRs for many components increase with the tension only braces. Most significant are the increases for the column at the base (0.83 to 1.70 DCR increase); beam at an intermediate level (0.52 to 1.90); brace connection at the tower base (0.66 to 1.22); beam at junction level (0.46 to 1.07); and beams at the base of the mullions (0.51 to 1.05). These values still fall below 2.0. Therefore, the seismic performance of the San Luis Obispo tower shaft is adequate and needs no upgrade.

Table 35. San Luis Obispo Shaft Evaluation Modal Analysis Results.

Mode #	SLO1, SLO2 & SLO3 Period (sec)	Cumulative Participating Mass (%) X-dir	Y-dir	Z-dir	Mode of Vibration
1	0.390	3.9	48.8	0.0	1st Y-Lateral
2	0.382	64.8	51.9	0.0	1st X-Lateral
3	0.266	65.0	51.9	0.0	1st Torsion
4	0.193	83.3	52.8	0.0	2nd X-Lateral
5	0.184	84.0	81.1	0.0	2nd Y-Lateral
6	0.142	84.6	81.6	0.0	2nd Torsion
7	0.135	84.6	81.6	7.4	1st Vertical – 4th floor/junction
8	0.127	85.4	82.2	9.2	2nd Vertical – 2nd floor
9	0.126	85.4	82.5	9.5	3rd Vertical – 2nd & 3rd floor
10	0.125	85.5	82.7	15.0	4th Vertical – 3rd floor
11	0.123	85.6	82.7	17.4	5th Vertical – 4th floor, cab roof/floor
12	0.116	86.4	82.7	21.3	6th Vertical – 2nd floor
15	0.112	86.6	84.4	28.1	7th Vertical –cab roof
16	0.108	92.7	85.7	28.1	3rd X-Lateral
17	0.100	94.0	89.7	28.2	3rd Y-Lateral
19	0.094	94.3	89.9	41.7	7th Vertical – cab floor & roof
24	0.083	95.1	90.3	51.1	9th Vertical – cab rocking

Table 36. San Luis Obispo Shaft Evaluation Selected Horizontal SRSS Deflections.

Location	SLO1 Joint #	SLO1 δ_{xe} (in.)	SLO2 Joint #	SLO2 δ_{xe} (in.)	SLO3 Joint #	SLO3 δ_{xe} (in.)
2nd Floor, δ_1	217	0.16	217	0.17	217	0.20
3rd Floor, δ_2	316	0.36	316	0.40	316	0.45
4th Floor/ Junction, δ_3	520	0.59	520	0.63	526	0.72
Top of Shaft, δ_4	824	1.13	824	1.20	824	1.32
Cab Floor, δ_5	920	1.76	920	1.81	920	1.82
Bottom of Windows	1008	2.14	1008	2.17	1012	2.15
Top of Windows	1108	2.61	1108	2.65	1112	2.63
Cab Roof, δ_6	1220	2.79	1220	2.84	1227	2.82
Parapet	1308	3.06	1308	3.12	1312	3.10

Table 37. Forces, Moments, and DCRs (SLO1, all windows, $\theta_L = 90°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	149.1	Gravity	-3.2	-0.1	0.0	0	0	0		
Base		104		H. Seismic	-47.3	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-1.0	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=101		Total	-51.5	-0.1	0.0	0	0	0	1.89	D1
Column at	S-4	C	118.5	Gravity	-45.6	0.0	0.0	0	0	-1		
Base		101		H. Seismic	-116.0	-0.7	-0.2	0	-16	-84		
				0.300 Gravity	-13.7	0.0	0.0	0	0	0		
(additive)	W8x31	n=101		Total	-175.3	-0.8	-0.2	0	-16	-85	0.70	D2
Beam at	S-2, S-4	I	22.6	Gravity	0.0	1.8	0.0	0	0	-69		
Intermediate	W14x34	210		H. Seismic	0.0	8.4	0.0	0	0	-759		
Levels	W16x40			0.300 Gravity	0.0	0.5	0.0	0	0	-21		
(additive)	W16x45	n=210		Total	0.0	10.7	0.0	0	0	-849	0.33	D3
Bracing conn	S-4	BR	-	Gravity	-3.2	-0.1	0.0	0	0	0		
at base		104		H. Seismic	47.3	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	1.0	0.0	0.0	0	0	0		
(counteract)		n=101		Total	45.0	-0.1	0.0	0	0	0	0.57	D4
Beam at	S-2, S-4	J	90.5	Gravity	0.0	-7.0	0.0	0	0	-34		
Jnct. level	W14x34	512		H. Seismic	0.0	-7.1	0.0	0	0	-781		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-2.1	0.0	0	0	-10		
(additive)	W16x45	n=511		Total	0.0	-16.2	0.0	0	0	-825	0.32	D5
Bracing btwn	S-4	BR	102.3	Gravity	-4.8	0.0	0.0	0	0	0		
Jnct & Wlkwy		503		H. Seismic	-52.2	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-1.4	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=503		Total	-58.4	0.0	0.0	0	0	0	1.57	D6
Beams at	S-2, S-3	TS	82.6	Gravity	0.6	0.9	0.0	0	-2	117		
Base of	W8x35,	807		H. Seismic	8.5	6.2	-1.9	1	-152	511		
mullions	W10x26			0.300 Gravity	0.2	0.3	0.0	0	-1	35		
(additive)	W16x77	n=808		Total	9.2	7.4	-1.9	1	-154	663	0.81	D7
Corner mullion	S-3	MUL	40.8	Gravity	-6.2	-0.1	-0.6	0	-23	-8		
connections		804		H. Seismic	9.5	-1.8	-13.5	-49	-454	-110		
at base				0.300 Gravity	1.9	0.0	-0.2	0	-7	-3		
(counteract)		n=824		Total	5.1	-1.9	-14.3	-49	-484	-121	1.93	D8
Mullions at	S-5	MUL	40.8	Gravity	-6.2	-0.1	-0.6	0	-23	-8		
top & btm of		804		H. Seismic	-9.5	-1.8	-13.5	-49	-454	-110		
window	TS			0.300 Gravity	-1.9	0.0	-0.2	0	-7	-3		
(additive)	8x4x1/2	n=824		Total	-17.5	-1.9	-14.3	-49	-484	-121	0.84	D9
Mullions	S-5	MUL	94.8	Gravity	-2.8	-0.3	-0.1	0	-8	-13		
within		1002		H. Seismic	-5.8	-3.4	-5.8	1	-296	-300		
window span	TS			0.300 Gravity	-0.8	-0.1	0.0	0	-2	-4		
(additive)	8x4x1/2	n=1004		Total	-9.5	-3.8	-5.9	1	-307	-316	0.78	D10
Tubes at	S-5	BW	57.5	Gravity	-0.3	-0.2	-0.1	0	3	11		
base of		1007		H. Seismic	-9.6	-13.4	-2.3	6	141	377		
window	TS			0.300 Gravity	-0.1	-0.1	0.0	0	1	3		
(additive)	7x7 3/16	n=1006		Total	-10.0	-13.7	-2.4	6	145	390	1.10	D11
Tubes at	S-5	TW	72.4	Gravity	0.3	0.0	0.0	0	0	6		
top of		1107		H. Seismic	6.9	-6.6	1.9	1	-73	237		
window	TS			0.300 Gravity	0.1	0.0	0.0	0	0	2		
(additive)	7x7 3/16	n=1106		Total	7.3	-6.6	1.9	1	-74	244	0.62	D12

Table 38. Forces, Moments, and DCRs (SLO2, all windows, θ_L = 45°, S_{DS} = 1.00g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
Base		108		H. Seismic	-40.6	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		Total	-50.2	-0.1	0.0	0	0	0	1.81	D1
Column at	S-4	C	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
Base		104		H. Seismic	-99.5	-0.6	-0.3	0	-33	-73		
				0.300 Gravity	-16.0	0.0	0.0	0	0	0		
(additive)	W8x31	n=104		Total	-168.8	-0.7	-0.3	0	-33	-74	0.70	D2
Beam at	S-2, S-4	I	50.0	Gravity	0.0	0.2	0.0	0	0	-40		
Intermediate	W14x34	223		H. Seismic	0.0	25.3	0.0	0	0	-654		
Levels	W16x40			0.300 Gravity	0.0	0.1	0.0	0	0	-12		
(additive)	W16x45	n=220		Total	0.0	25.6	0.0	0	0	-707	0.37	D3
Bracing conn	S-4	BR	-	Gravity	-7.4	-0.1	0.0	0	0	0		
at base		108		H. Seismic	40.6	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	2.2	0.0	0.0	0	0	0		
(counteract)		n=105		Total	35.3	-0.1	0.0	0	0	0	0.44	D4
Beam at	S-2, S-4	J	90.5	Gravity	0.0	-7.0	0.0	0	0	-34		
Jnct. level	W14x34	512		H. Seismic	0.0	-8.5	0.0	0	0	-702		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-2.1	0.0	0	0	-10		
(additive)	W16x45	n=511		Total	0.0	-17.7	0.0	0	0	-746	0.29	D5
Bracing btwn	S-4	BR	120.0	Gravity	-9.4	0.0	0.0	0	0	0		
Jnct & Wlkwy		508		H. Seismic	-48.1	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.8	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		Total	-60.3	-0.1	0.0	0	0	0	1.84	D6
Beams at	S-2, S-3	TS	82.04	Gravity	0.6	0.9	0.0	0	-2	117		
Base of	W8x35,	807		H. Seismic	10.7	5.7	-1.7	1	-140	467		
mullions	W10x26			0.300 Gravity	0.2	0.3	0.0	0	-1	35		
(additive)	W16x77	n=808		Total	11.4	6.8	-1.7	1	-143	619	0.76	D7
Corner mullion	S-3	MUL	40.75	Gravity	-6.2	-0.1	-0.6	0	-23	-8		
connections		804		H. Seismic	14.2	-3.0	-12.4	-45	-428	-105		
at base				0.300 Gravity	1.9	0.0	-0.2	0	-7	-3		
(counteract)		n=824		Total	9.8	-3.1	-13.2	-45	-458	-116	1.86	D8
Mullions at	S-5	MUL	37.5	Gravity	-7.0	0.4	0.0	0	-1	0		
top & btm of		906		H. Seismic	-21.7	13.1	3.0	-8	-189	-630		
window	TS			0.300 Gravity	-2.1	0.1	0.0	0	0	0		
(additive)	8x4x1/2	n=1012		Total	-30.7	13.6	3.1	-8	-190	-631	0.88	D9
Mullions	S-5	MUL	94.63	Gravity	-5.0	-0.5	-0.1	0	-3	-25		
within		1006		H. Seismic	-9.4	-4.5	-4.4	2	-229	-414		
window span	TS			0.300 Gravity	-1.5	-0.1	0.0	0	-1	-7		
(additive)	8x4x1/2	n=1012		Total	-16.0	-5.1	-4.5	2	-234	-446	0.83	D10
Tubes at	S-5	BW	57.5	Gravity	-0.2	0.5	-0.1	0	6	-8		
base of		1005		H. Seismic	-8.6	12.0	-2.5	5	125	-352		
window	TS			0.300 Gravity	-0.1	0.1	0.0	0	2	-2		
(additive)	7x7 3/16	n=1004		Total	-8.9	12.6	-2.6	5	133	-363	1.01	D11
Tubes at	S-5	TW	72.4	Gravity	0.5	-0.2	-0.1	0	-2	-4		
top of		1109		H. Seismic	8.9	-6.0	-1.4	1	-66	-215		
window	TS			0.300 Gravity	0.2	-0.1	0.0	0	-1	-1		
(additive)	7x7 3/16	n=1110		Total	9.6	-6.3	-1.5	1	-68	-220	0.56	D12

Table 39. Forces, Moments, and DCRs (SLO3, all windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	149.1	Gravity	-7.6	-0.1	0.0	0	0	0		
Base		107		H. Seismic	-57.9	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.3	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=104		Total	-67.8	-0.1	0.0	0	0	0	3.06	D1
Column at	S-4	C	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
Base		104		H. Seismic	-136.7	-0.3	-0.4	0	-45	-37		
				0.300 Gravity	-16.0	0.0	0.0	0	0	0		
(additive)	W8x31	n=104		Total	-206.0	-0.4	-0.4	0	-45	-38	0.83	D2
Beam at	S-2, S-4	I	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
Intermediate	W14x34	223		H. Seismic	0.0	-36.2	0.0	0	0	-921		
Levels	W16x40			0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
(additive)	W16x45	n=219		Total	0.0	-37.3	0.0	0	0	-994	0.52	D3
Bracing conn	S-4	BR	-	Gravity	-7.6	0.1	0.0	0	0	0		
at base		107		H. Seismic	57.9	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	2.3	0.0	0.0	0	0	0		
(counteract)		n=104		Total	52.6	0.1	0.0	0	0	0	0.66	D4
Beam at	S-2, S-4	J	50.0	Gravity	0.0	-2.1	0.0	0	0	-78		
Jnct. level	W14x34	527		H. Seismic	0.0	-29.8	0.0	0	0	-769		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
(additive)	W16x45	n=523		Total	0.0	-32.5	0.0	0	0	-870	0.46	D5
Bracing btwn	S-4	BR	120.0	Gravity	-9.4	0.0	0.0	0	0	0		
Jnct & Wlkwy		508		H. Seismic	-70.1	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.8	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		Total	-82.2	-0.1	0.0	0	0	0	2.91	D6
Beams at	S-2, S-3	TS	43.6	Gravity	-3.6	-7.7	-0.2	0	-12	-68		
Base of	W8x35,	833		H. Seismic	-8.3	-48.8	-7.3	-6	-287	-1486		
mullions	W10x26			0.300 Gravity	-1.1	-2.3	-0.1	0	-3	-20		
(additive)	W16x77	n=827		Total	-12.9	-58.9	-7.6	-6	-302	-1575	0.51	D7
Corner mullion	S-3	MUL	40.8	Gravity	-6.0	-0.1	0.7	-2	24	-6		
connections		805		H. Seismic	16.8	-2.2	14.9	-53	488	-131		
at base				0.300 Gravity	1.8	0.0	0.2	0	7	-2		
(counteract)		n=825		Total	12.6	-2.3	15.8	-55	519	-139	2.14	D8
Mullions at	S-5	MUL	37.5	Gravity	-3.3	0.1	-0.2	0	3	-4		
top & btm of		905		H. Seismic	-16.5	13.6	-3.1	-10	228	-455		
window	TS			0.300 Gravity	-1.0	0.0	-0.1	0	1	-1		
(additive)	8x4x1/2	n=1009		Total	-20.9	13.7	-3.3	-10	232	-460	0.77	D9
Mullions	S-5	MUL	94.6	Gravity	-5.0	-0.5	-0.1	0	-3	-25		
within		1006		H. Seismic	-5.0	-2.5	-6.3	1	-328	-235		
window span	TS			0.300 Gravity	-1.5	-0.1	0.0	0	-1	-7		
(additive)	8x4x1/2	n=1012		Total	-11.6	-3.1	-6.4	1	-332	-267	0.79	D10
Tubes at	S-5	BW	57.6	Gravity	-0.9	0.5	0.2	0	5	21		
base of		1012		H. Seismic	-9.5	13.2	1.2	-6	120	373		
window	TS			0.300 Gravity	-0.3	0.2	0.1	0	1	6		
(additive)	7x7 3/16	n=1011		Total	-10.7	13.9	1.4	-7	126	400	1.09	D11
Tubes at	S-5	TW	72.4	Gravity	0.4	0.3	0.1	0	-3	-6		
top of		1110		H. Seismic	10.5	6.5	1.5	-1	-67	-234		
window	TS			0.300 Gravity	0.1	0.1	0.0	0	-1	-2		
(additive)	7x7 3/16	n=1111		Total	11.0	6.9	1.6	-1	-70	-242	0.61	D12

Table 40. Braces Between Foundation and Junction Level (SLO3, all windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing btwn	S-4	BR	149.1	Gravity	-6.4							
Fndtn & I2		101		H. Seismic	-57.7							
L6x6x1/2				0.300 Gravity	-1.9	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=101		Total	-66.0	0.0	0.0	0	0	0	2.89	D1
Bracing btwn	S-4	BR	149.1	Gravity	-6.6							
Fndtn & I2		102		H. Seismic	-58.4							
L6x6x1/2				0.300 Gravity	-2.0	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=102		Total	-66.9	0.0	0.0	0	0	0	2.97	D1
Bracing btwn	S-4	BR	149.1	Gravity	-7.6							
Fndtn & I2		107		H. Seismic	-57.9							
L6x6x1/2				0.300 Gravity	-2.3	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=104		Total	-67.8	0.0	0.0	0	0	0	3.06	D1
Bracing btwn	S-4	BR	149.1	Gravity	-7.5							
Fndtn & I2		108		H. Seismic	-57.4							
L6x6x1/2				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=105		Total	-67.1	0.0	0.0	0	0	0	2.99	D1
Bracing btwn	S-4	BR	150.3	Gravity	-6.3							
I2 & I3		201		H. Seismic	-54.4							
L6x6x1/2				0.300 Gravity	-1.9	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=201		Total	-62.6	0.0	0.0	0	0	0	2.61	D1
Bracing btwn	S-4	BR	150.3	Gravity	-6.2							
I2 & I3		202		H. Seismic	-54.7							
L6x6x1/2				0.300 Gravity	-1.9	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=205		Total	-62.7	0.0	0.0	0	0	0	2.62	D1
Bracing btwn	S-4	BR	150.3	Gravity	-7.2							
I2 & I3		207		H. Seismic	-58.5							
L6x6x1/2				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=217		Total	-67.9	0.0	0.0	0	0	0	3.07	D1
Bracing btwn	S-4	BR	150.3	Gravity	-7.3							
I2 & I3		208		H. Seismic	-58.1							
L6x6x1/2				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=221		Total	-67.6	0.0	0.0	0	0	0	3.04	D1
Bracing btwn	S-4	BR	150.3	Gravity	-13.1							
I3 & JNCTN		401		H. Seismic	-45.7							
L6x6x1/2				0.300 Gravity	-3.9	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=301		Total	-62.7	0.0	0.0	0	0	0	2.62	D1
Bracing btwn	S-4	BR	150.3	Gravity	-12.0							
I3 & JNCTN		402		H. Seismic	-53.8							
L6x6x1/2				0.300 Gravity	-3.6	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=305		Total	-69.4	0.0	0.0	0	0	0	3.22	D1
Bracing btwn	S-4	BR	150.3	Gravity	-14.4							
I3 & JNCTN		407		H. Seismic	-58.9							
L6x6x1/2				0.300 Gravity	-4.3	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=316		Total	-77.6	0.0	0.0	0	0	0	4.25	D1
Bracing btwn	S-4	BR	150.3	Gravity	-15.5							
I3 & JNCTN		408		H. Seismic	-48.1							
L6x6x1/2				0.300 Gravity	-4.7	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=320		Total	-68.3	0.0	0.0	0	0	0	3.11	D1

Table 41. Braces Between Junction and Top of Shaft (SLO3, all windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing btwn	S-4	BR	103.0	Gravity	-7.3							
JNCTN&CBACC		501		H. Seismic	-46.9							
L6x6x1/2				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=501		Total	-56.5	0.0	0.0	0	0	0	1.51	D1
Bracing btwn	S-4	BR	101.4	Gravity	-8.1							
JNCTN&CBACC		502		H. Seismic	-48.1							
L6x6x1/2				0.300 Gravity	-2.4	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=506		Total	-58.6	0.0	0.0	0	0	0	1.58	D1
Bracing btwn	S-4	BR	98.7	Gravity	-10.9							
JNCTN&CBACC		507		H. Seismic	-66.0							
L6x6x1/2				0.300 Gravity	-3.3	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=520		Total	-80.1	0.0	0.0	0	0	0	2.35	D1
Bracing btwn	S-4	BR	120.05	Gravity	-9.4							
JNCTN&CBACC		508		H. Seismic	-70.1							
L6x6x1/2				0.300 Gravity	-2.8	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=526		Total	-82.2	0.0	0.0	0	0	0	2.91	D1
Bracing btwn	S-4	BR	90.5	Gravity	-7.1							
CBACC&TofS		701		H. Seismic	-47.0							
L6x6x1/2				0.300 Gravity	-2.1	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=602		Total	-56.2	0.0	0.0	0	0	0	1.40	D1
Bracing btwn	S-4	BR	89.1	Gravity	-7.9							
CBACC&TofS		702		H. Seismic	-48.0							
L6x6x1/2				0.300 Gravity	-2.4	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=605		Total	-58.2	0.0	0.0	0	0	0	1.46	D1
Bracing btwn	S-4	BR	86.8	Gravity	-10.6							
CBACC&TofS		707		H. Seismic	-65.9							
L6x6x1/2				0.300 Gravity	-3.2	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=614		Total	-79.7	0.0	0.0	0	0	0	2.06	D1
Bracing btwn	S-4	BR	70.5	Gravity	-8.6							
CBACC&TofS		708		H. Seismic	-59.1							
L6x6x1/2				0.300 Gravity	-2.6	0.0	0.0	0	0	0	*	
(Buckling)	L6x6x1/2	n=710		Total	-70.3	0.0	0.0	0	0	0	0.12	D1

* Low DCR because the brace length is much less.

Table 42. Tension Braces Only (SLO3a, all windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing btwn	S-4	BR	149.1	Gravity	0.8							
Fndtn & I2		102		H. Seismic	95.9							
L6x6x1/2				0.300 Gravity	0.2	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=102		Total	97.0	0.0	0.0	0	0	0	1.85	D1a
Bracing btwn	S-4	BR	149.1	Gravity	-5.0							
Fndtn & I2		108		H. Seismic	66.6							
L6x6x1/2				-0.300 Gravity	1.5	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=105		Total	63.1	0.0	0.0	0	0	0	1.21	D1a
Bracing btwn	S-4	BR	150.3	Gravity	3.3						2.09	D1a
I2 & I3		202		H. Seismic	105.1						(w/bending)	
L6x6x1/2				0.300 Gravity	1.0	0.0	0.0	0	0	0	0.53	
(Tension)	L6x6x1/2	n=205		Total	109.4	0.0	0.0	0	0	0	(wo/bending)	
Bracing btwn	S-4	BR	150.3	Gravity	-2.8							
I2 & I3		208		H. Seismic	75.2							
L6x6x1/2				-0.300 Gravity	0.8	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=221		Total	73.3	0.0	0.0	0	0	0	1.40	D1a
Bracing btwn	S-4	BR	150.3	Gravity	4.0							
I3 & JNCTN		402		H. Seismic	88.6							
L6x6x1/2				0.300 Gravity	1.2	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=305		Total	93.7	0.0	0.0	0	0	0	1.79	D1a
Bracing btwn	S-4	BR	150.3	Gravity	-3.7							
I3 & JNCTN		408		H. Seismic	77.3							
L6x6x1/2				-0.300 Gravity	1.1	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=320		Total	74.7	0.0	0.0	0	0	0	1.43	D1a
Bracing btwn	S-4	BR	101.4	Gravity	-0.6							
JNCTN&CBACC		502		H. Seismic	68.9							
L6x6x1/2				-0.300 Gravity	0.2	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=506		Total	68.4	0.0	0.0	0	0	0	1.31	D1a
Bracing btwn	S-4	BR	120.05	Gravity	-1.0							
JNCTN&CBACC		508		H. Seismic	84.0							
L6x6x1/2				-0.300 Gravity	0.3	0.0	0.0	0	0	0		
(Tension)	L6x6x1/2	n=526		Total	83.3	0.0	0.0	0	0	0	1.59	D1a
Bracing btwn	S-4	BR	89.1	Gravity	-0.4							
CBACC&TofS		702		H. Seismic	68.9							
L6x6x1/2				-0.300 Gravity	0.1	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=605		Total	68.6	0.0	0.0	0	0	0	1.31	D1a
Bracing btwn	S-4	BR	70.5	Gravity	-0.5							
CBACC&TofS		708		H. Seismic	76.1							
L6x6x1/2				-0.300 Gravity	0.2	0.0	0.0	0	0	0		
(Buckling)	L6x6x1/2	n=710		Total	75.7	0.0	0.0	0	0	0	1.45	D1a

Table 43. Other Components w/Tension Braces Only (SLO3a, all windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	-	Gravity	0.8	0.0	0.0	0	0	0		
Base		102		H. Seismic	95.9	0.0	0.0	0	0	0		
- Tension				0.300 Gravity	0.2	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=102		Total	97.0	0.0	0.0	0	0	0	1.85	D1a
Column at	S-4	C	118.5	Gravity	-53.4	0.0	0.6	0	59	-3		
Base		104		H. Seismic	-98.4	-0.3	4.3	0	459	-34		
				0.300 Gravity	-16.0	0.0	0.2	0	18	-1		
(additive)	W8x31	n=104		Total	-167.9	-0.3	5.1	0	536	-38	1.70	D2
Beam at	S-2, S-4	I	50.0	Gravity	0.0	0.0	0.0	0	0	440		
Intermediate	W14x34	203		H. Seismic	0.0	29.9	0.0	0	0	4261		
Levels	W16x40			0.300 Gravity	0.0	0.0	0.0	0	0	138		
(additive)	W16x45	n=204		Total	0.0	29.9	0.0	0	0	4840	1.90	D3
Bracing conn	S-4	BR	-	Gravity	0.6	-0.1	0.0	0	0	0		
at base		102		H. Seismic	95.9	0.0	0.0	0	0	0		
- Tension				0.300 Gravity	0.2	0.0	0.0	0	0	0		
(additive)		n=102		Total	96.7	-0.1	0.0	0	0	0	1.22	D4
Beam at	S-2, S-4	J	102.3	Gravity	0.0	-1.3	0.0	0	0	448		
Jnct. level	W14x34	503		H. Seismic	0.0	-24.6	0.0	0	0	2146		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-0.4	0.0	0	0	134		
(additive)	W16x45	n=504		Total	0.0	-26.3	0.0	0	0	2729	1.07	D5
Bracing btwn	S-4	BR	-	Gravity	3.3	0.0	0.0	0	0	0		
Jnct & Wlkwy		202		H. Seismic	105.1	0.0	0.0	0	0	0		
- Tension				0.300 Gravity	1.0	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=205		Total	109.4	0.0	0.0	0	0	0	2.09	D1a
Beams at	S-2, S-3	TS	75.1	Gravity	0.9	1.3	0.6	1	-19	864		
Base of	W8x35,	834		H. Seismic	27.1	55.3	2.5	6	-59	4134		
mullions	W10x26			0.300 Gravity	0.3	0.4	0.2	0	-6	259		
(additive)	W16x77	n=829		Total	28.2	57.0	3.2	7	-83	5257	1.05	D7
Corner mullion	S-3	MUL	40.75	Gravity	-1.9	-1.3	0.0	0	-1	51		
connections		806		H. Seismic	38.7	-4.4	11.9	17	-97	168		
at base				0.300 Gravity	0.6	-0.4	0.0	0	0	15		
(counteract)		n=828		Total	37.4	-6.1	11.9	17	-98	235	1.07	D8
Mullions at	S-5	MUL	37.5	Gravity	-8.2	-0.3	0.2	-1	-10	0		
top & btm of		902		H. Seismic	-26.3	-6.1	4.4	-6	-159	290		
window	TS			0.300 Gravity	-2.5	-0.1	0.1	0	-3	0		
(additive)	8x4x1/2	n=1004		Total	-37.0	-6.4	4.7	-6	-172	290	0.56	D9
Mullions	S-5	MUL	94.8	Gravity	-4.6	-0.7	1.1	0	69	-46		
within		1005		H. Seismic	-4.6	-4.0	3.9	1	249	-337		
window span	TS			0.300 Gravity	-1.4	-0.2	0.3	0	21	-14		
(additive)	8x4x1/2	n=1009		Total	-10.6	-4.9	5.4	1	339	-397	0.91	D10
Tubes at	S-5	BW	57.6	Gravity	2.2	2.8	0.6	-1	-36	-74		
base of		1011		H. Seismic	12.7	17.4	4.4	-3	-182	-495		
window	TS			0.300 Gravity	0.7	0.8	0.2	0	-11	-22		
(additive)	7x7 3/16	n=1010		Total	15.6	21.0	5.2	-4	-229	-591	1.57	D11
Tubes at	S-5	TW	72.4	Gravity	-1.8	1.5	0.0	0	-25	-50		
top of		1111		H. Seismic	-10.9	9.1	0.8	1	-131	-323		
window	TS			0.300 Gravity	-0.5	0.5	0.0	0	-7	-15		
(additive)	7x7 3/16	n=1110		Total	-13.2	11.0	0.8	1	-163	-388	1.23	D12

Cab Members and Connections

The cab members and connections of the San Luis Obispo tower were evaluated with cases SLO4, SLO5, SLO6, and SLO7. Case SLO4 is for one window acting as a shear wall and cases SLO5 through SLO7 represent the most realistic condition where no windows act as a shear wall. The following sections summarize the San Luis Obispo cab evaluation in terms of modal behavior, deflections, story drifts, and component forces and moments used to calculate DCRs.

The SLO4 case assumes the window acting as a shear wall is located farthest from the center of mass (for 5 percent accidental eccentricity), which is placed farthest from the plane of this shear wall window. This will create the greatest distance between the center of mass and center of stiffness in the plan of the building and will create the greatest torsional response of the tower shaft. The resultant direction of the 100 percent full seismic spectrum loading plus the 30 percent orthogonal spectrum is parallel to this shear wall window, to create the worst torsional response.

Modal Analysis Results and Deflections

Table 44 presents the primary modes of vibration for the San Luis Obispo tower, evaluated in the SLO4, SLO5, SLO6, and SLO7 analysis cases. The cumulative participating mass shows that a smaller portion of the mass participates in the first X and Y lateral modes than in the cases where cab windows act as shear walls (SLO1, SLO2, and SLO3 in Table 35).

Table 45 presents the maximum lateral deflections at each floor level and other key locations in the San Luis Obispo control tower. Table 46 shows the greatest San Luis Obispo story drifts, which were for Case SLO7 (θ_L = 0°) for shaft and Case SLO5 (θ_L = 90°) for the cab. The calculated story drifts, Δ_c, fall below the allowable values, Δ_a, at each floor level. Therefore, no stiffening upgrade is needed to reduce deflections and story drifts.

Cab Member and Connection Evaluation

Tables 47 through 50 give the force and moment summary for the most critically stressed members and connections. These tables also present the DCRs for each of these critical components. Case SLO7 (Table 50), with the full seismic spectrum at 0 degrees to the X-axis (θ_L = 0°), is the most critical loading condition for the cab members and connections. The highest DCR is for the cab column connections (window corner mullions). Table 50 shows that the most heavily loaded

Table 44. San Luis Obispo Cab Evaluation Modal Analysis Results.

Mode #	SLO5, SLO6, and SLO7 Period (sec)	Cumulative Participating Mass (%) X-dir	Y-dir	Z-dir	Mode of Vibration	SLO4 Period (sec)
1	0.499	0.1	32.4	0.0	1st Y-Lateral	0.483
2	0.465	40.3	32.4	0.0	1st X-Lateral	0.441
3	0.338	40.3	32.4	0.0	1st Torsion	0.299
4	0.255	80.7	32.7	0.0	2nd X-Lateral	0.242
5	0.234	81.0	75.0	0.0	2nd Y-Lateral	0.227
6	0.165	81.7	75.4	0.0	2nd Torsion	0.150
7	0.135	81.8	75.4	7.4	1st Vertical – 4th floor/junction	0.135
8	0.132	82.1	77.9	7.4	3rd Y-Lateral	
9	0.127	83.3	79.4	10.8	2nd Vertical – 2nd & 3rd floor	0.127
10	0.126	83.3	86.4	11.1	3rd Y-Lateral	0.123
11	0.125	83.3	86.7	13.0	3rd Vertical – 2nd & 3rd floor	0.125
12	0.123	83.3	86.7	16.1	4th Vertical – 4th floor, cab roof/floor	0.123
13	0.119	89.0	86.7	20.6	3rd X-Lateral & Vertical – 2nd floor	0.118
14	0.115	89.1	86.7	20.8	5th Vertical – 2nd and 3rd floor	0.115
15	0.113	91.7	86.8	23.4	3rd X-Lateral – cab rocking	0.113
16	0.112	91.7	86.8	28.0	5th Vertical –cab roof	0.112
20	0.094	93.4	88.5	33.7	6th Vertical – cab floor	0.093
21	0.093	93.4	88.5	43.3	7th Vertical – cab floor & roof	0.092
23	0.089	94.9	89.3	43.6	4th X-Lateral	0.087
25	0.084	95.3	89.5	47.0	8th Vertical – cab access stair support	
26	0.083	95.3	91.7	50.2	9th Vertical/Y-Lateral – cab rocking	0.083

Table 45. San Luis Obispo Cab Evaluation Selected Horizontal SRSS Deflections.

Location	SLO4 Joint #	SLO4 δ_{xe} (in.)	SLO5 Joint #	SLO5 δ_{xe} (in.)	SLO6 Joint #	SLO6 δ_{xe} (in.)	SLO7 Joint #	SLO7 δ_{xe} (in.)
2nd Floor, δ_1	217	0.18	217	0.13	217	0.16	217	0.18
3rd Floor, δ_2	316	0.40	316	0.30	316	0.36	316	0.40
4th Floor/ Junction, δ_3	520	0.62	520	0.49	520	0.56	520	0.62
Top of Shaft, δ_4	824	1.11	804	0.94	824	1.01	824	1.09
Cab Floor, δ_5	922	1.63	920	1.64	920	1.60	920	1.56
Bottom of Windows	1012	2.28	1008	2.22	1008	2.16	1012	2.10
Top of Windows	1112	4.28	1108	3.93	1108	3.87	1112	3.87
Cab Roof, δ_6	1227	4.70	1220	4.39	1220	4.34	1227	4.35
Parapet	1312	5.27	1308	5.02	1308	4.98	1312	5.00

of these connections has a DCR of 2.49. The TS 8 x 4 x ½ structural tubing mullion is welded to a 1-1/8-in.-thick base plate with a 5/16-in. fillet weld (San Luis Obispo tower Drawing No. S-5; Leo A Daly 1981). The critical mode of failure is shearing through the throat of the weld (see Appendix D8). The axial load in the tubing due to horizontal and vertical seismic load could be either positive (tensile) or negative, but it is assumed to be positive, which counteracts the

effects of gravity. This is because only tensile forces at this joint will load the weld in shear. Compressive force would simply transfer the load through bearing, not loading the critical weld. Only the connections of these mullions have DCRs greater than 2.0.

If one mullion connection begins to fail, loads would be redistributed to the other mullions because the mullions will all deflect together, as they are tied to each other at the cab floor. Therefore, the condition of all the mullion connections at their base must be examined and, if their average DCR exceeds 2.0, failure will begin. Table 51 gives the forces, moments, and DCRs for all the mullion connections to their base plates. The average DCR for all mullions is 1.97 as shown in the table. This is slightly less than the 2.0 value that defines failure. Weld failure would be brittle and must be prevented, so the design resistance factors are conservatively left in the evaluation (Appendix D8).

Channels that support the catwalk are connected (shear connections) to these mullions all the way around the cab perimeter. These channels are located just above the critical mullion base connections, but are not included in the San Luis Obispo model. The cab floor beams also frame into the mullions at a higher level with shear connections. Both the catwalk channels and cab floor beams will provide some moment resistance, reducing rotations and deflections at these levels and reducing the moment applied to the mullion base connections.

The design resistance factors remaining in the mullion connection evaluation and the catwalk channels and cab floor beam connections all reduce the stress condition of the critical mullion connections, and these effects were not accounted for in this evaluation. These unaccounted for contributions further ensure that the critical mullion connections (DCR = 1.97) will not fail in the maximum considered earthquake motions. Therefore, this tower has met the requirements of the life-safety evaluation and no upgrade is needed.

Table 46. San Luis Obispo Tower Story Drifts.

Location	Story Elev Y_x (in.)	Story Height $h_{sx} = Y_x - Y_{x-1}$ (in.)	Allow Story Drift Δ_a (in.)	Analysis Case	Joint #	Elastic Defl δ_x (in.)	Calc Story Drift Δ_c (in.)
Ground Floor	-9.5			SLO7	104	0.00	
2nd Floor	109	118.5	2.37	SLO7	217	0.18	0.18
3rd Floor	229	120	2.4	SLO7	316	0.40	0.22
4th Floor	349	120	2.4	SLO7	520	0.62	0.22
Shaft Roof	520	171	3.42	SLO7	826	1.03	0.41
Shaft Roof	520	171	3.42	SLO5	824	0.92	
Cab Floor	560.8	40.75	0.815	SLO5	918	1.56	0.64
Cab Roof	718.8	158	3.16	SLO5	1220	4.39	2.83

Table 47. Forces, Moments, and DCRs (SLO4, one window, $\theta_L = 16.7°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
Base		108		H. Seismic	-49.4	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		Total	-59.1	-0.1	0.0	0	0	0	2.36	D1
Column at	S-4	C	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
Base		104		H. Seismic	-81.1	-0.5	0.4	0	39	-51		
				0.300 Gravity	-16.0	0.0	0.0	0	0	0		
(additive)	W8x31	n=104		Total	-150.3	-0.5	0.4	0	39	-52	0.63	D2
Beam at	S-2, S-4	I	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
Intermediate	W14x34	223		H. Seismic	0.0	-30.9	0.0	0	0	-767		
Levels	W16x40			0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
(additive)	W16x45	n=219		Total	0.0	-32.0	0.0	0	0	-840	0.44	D3
Bracing conn	S-4	BR	-	Gravity	-7.4	-0.1	0.0	0	0	0		
at base		108		H. Seismic	49.4	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	2.2	0.0	0.0	0	0	0		
(counteract)		n=105		Total	44.2	-0.1	0.0	0	0	0	0.56	D4
Beam at	S-2, S-4	J	50.0	Gravity	0.0	-2.1	0.0	0	0	-77		
Jnct. level	W14x34	527		H. Seismic	0.0	-25.3	0.0	0	0	-636		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
(additive)	W16x45	n=523		Total	0.0	-28.0	0.0	0	0	-737	0.39	D5
Bracing btwn	S-4	BR	120.0	Gravity	-9.1	0.0	0.0	0	0	0		
Jnct & Wlkwy		508		H. Seismic	-62.8	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.7	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		Total	-74.6	-0.1	0.0	0	0	0	2.49	D6
Beams at	S-2, S-3	TS	43.6	Gravity	-3.4	-7.4	-0.2	0	-12	-65		
Base of	W8x35,	833		H. Seismic	-8.1	-42.4	-5.2	-4	-276	-1301		
mullions	W10x26			0.300 Gravity	-1.0	-2.2	-0.1	0	-4	-20		
(additive)	W16x77	n=827		Total	-12.6	-51.9	-5.5	-5	-292	-1385	0.47	D7
Corner mullion	S-3	MUL	40.8	Gravity	-11.5	-0.4	-0.1	0	-2	-1		
connections		806		H. Seismic	6.7	-1.2	-20.5	-26	-800	-9		
at base				0.300 Gravity	3.5	-0.1	0.0	0	-1	0		
(counteract)		n=828		Total	-1.4	-1.7	-20.6	-26	-803	-11	2.73	D8
Mullions at	S-5	MUL	40.8	Gravity	-11.5	-0.4	-0.1	0	-2	-1		
top & btm of		806		H. Seismic	-6.7	-1.2	-20.5	-26	-800	-9		
window	TS			0.300 Gravity	-3.5	-0.1	0.0	0	-1	0		
(additive)	8x4x1/2	n=828		Total	-21.6	-1.7	-20.6	-26	-803	-11	1.21	D9
Mullions	S-5	MUL	94.6	Gravity	-4.7	-0.4	0.0	0	-2	-18		
within		1006		H. Seismic	-2.3	-2.0	-8.7	35	-435	-105		
window span	TS			0.300 Gravity	-1.4	-0.1	0.0	0	-1	-5		
(additive)	8x4x1/2	n=1012		Total	-8.5	-2.6	-8.8	35	-437	-128	0.81	D10
Tubes at	S-5	BW	57.6	Gravity	-0.8	0.5	0.0	-1	0	-6		
base of		1009		H. Seismic	-1.0	10.0	1.1	-12	-54	-567		
window	TS			0.300 Gravity	-0.2	0.1	0.0	0	0	-2		
(additive)	7x7 3/16	n=1008		Total	-2.0	10.6	1.1	-14	-55	-574	1.23	D11
Tubes at	S-5	TW	72.4	Gravity	0.8	0.3	0.0	0	1	-5		
top of		1109		H. Seismic	1.1	5.2	-0.5	-7	34	-368		
window	TS			0.300 Gravity	0.2	0.1	0.0	0	0	-1		
(additive)	7x7 3/16	n=1108		Total	2.0	5.6	-0.6	-8	35	-374	0.79	D12

Table 48. Forces, Moments, and DCRs (SLO5, no windows, $\theta_L = 90°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	146.1	Gravity	-1.3	-0.1	0.0	0	0	0		
Base		103		H. Seismic	-46.3	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-0.4	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=102		Total	-48.0	-0.1	0.0	0	0	0	1.70	D1
Column at	S-4	C	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
Base		104		H. Seismic	-81.9	-0.7	0.1	0	-15	-79		
				0.300 Gravity	-16.0	0.0	0.0	0	0	0		
(additive)	W8x31	n=104		Total	-151.2	-0.7	0.1	0	-15	-80	0.61	D2
Beam at	S-2, S-4	I	22.6	Gravity	0.0	1.8	0.0	0	0	-69		
Intermediate	W14x34	210		H. Seismic	0.0	7.4	0.0	0	0	-677		
Levels	W16x40			0.300 Gravity	0.0	0.5	0.0	0	0	-21		
(additive)	W16x45	n=210		Total	0.0	9.7	0.0	0	0	-767	0.30	D3
Bracing conn	S-4	BR	-	Gravity	-1.3	-0.1	0.0	0	0	0		
at base		103		H. Seismic	46.3	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	0.4	0.0	0.0	0	0	0		
(counteract)		n=102		Total	45.4	0.0	0.0	0	0	0	0.57	D4
Beam at	S-2, S-4	J	50.0	Gravity	0.0	-0.6	0.0	0	0	-32		
Jnct. level	W14x34	513		H. Seismic	0.0	-27.9	0.0	0	0	-712		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-0.2	0.0	0	0	-10		
(additive)	W16x45	n=511		Total	0.0	-28.7	0.0	0	0	-754	0.41	D5
Bracing btwn	S-4	BR	102.3	Gravity	-5.0	0.0	0.0	0	0	0		
Jnct & Wlkwy		503		H. Seismic	-45.7	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-1.5	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=501		Total	-52.2	0.0	0.0	0	0	0	1.38	D6
Beams at	S-2, S-3	TS	82.0	Gravity	0.6	1.0	0.0	0	-2	129		
Base of	W8x35,	807		H. Seismic	6.1	6.7	-1.3	1	-103	547		
mullions	W10x26			0.300 Gravity	0.2	0.3	0.0	0	-1	39		
(additive)	W16x77	n=808		Total	6.9	8.0	-1.3	1	-105	714	0.76	D7
Corner mullion	S-3	MUL	40.8	Gravity	-5.7	-0.1	-0.6	1	-20	-5		
connections		803		H. Seismic	8.9	-2.9	-13.8	72	-503	-132		
at base				0.300 Gravity	1.7	0.0	-0.2	0	-6	-2		
(counteract)		n=809		Total	4.9	-3.1	-14.5	73	-530	-139	2.13	D8
Mullions at	S-5	MUL	40.8	Gravity	-5.7	-0.1	-0.6	1	-20	-5		
top & btm of		803		H. Seismic	-8.9	-2.9	-13.8	72	-503	-132		
window	TS			0.300 Gravity	-1.7	0.0	-0.2	0	-6	-2		
(additive)	8x4x1/2	n=809		Total	-16.3	-3.1	-14.5	73	-530	-139	0.92	D9
Mullions	S-5	MUL	94.8	Gravity	-3.5	-0.3	0.1	0	3	-14		
within		1004		H. Seismic	-2.7	-3.3	6.8	5	337	-162		
window span	TS			0.300 Gravity	-1.1	-0.1	0.0	0	1	-4		
(additive)	8x4x1/2	n=1008		Total	-7.3	-3.7	6.9	5	341	-180	0.70	D10
Tubes at	S-5	BW	57.5	Gravity	-0.8	-0.6	0.0	0	-1	-12		
base of		1007		H. Seismic	-3.6	-10.7	-1.1	39	-55	-595		
window	TS			0.300 Gravity	-0.2	-0.2	0.0	0	0	-4		
(additive)	7x7 3/16	n=1008		Total	-4.6	-11.5	-1.2	40	-57	-610	1.33	D11
Tubes at	S-5	TW	72.4	Gravity	0.8	0.3	0.0	0	1	-6		
top of		1105		H. Seismic	1.5	5.4	-0.7	15	42	-369		
window	TS			0.300 Gravity	0.2	0.1	0.0	0	0	-2		
(additive)	7x7 3/16	n=1104		Total	2.5	5.8	-0.7	15	43	-376	0.81	D12

Table 49. Forces, Moments, and DCRs (SLO6, no windows, $\theta_L = 45°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
Base		108		H. Seismic	-40.0	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		Total	-49.7	-0.1	0.0	0	0	0	1.788	D1
Column at	S-4	C	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
Base		104		H. Seismic	-88.3	-0.5	0.3	0	-32	-59		
				0.300 Gravity	-16.0	0.0	0.0	0	0	0		
(additive)	W8x31	n=104		Total	-157.6	-0.5	0.3	0	-32	-60	0.651	D2
Beam at	S-2, S-4	I	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
Intermediate	W14x34	223		H. Seismic	0.0	-25.1	0.0	0	0	-624		
Levels	W16x40			0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
(additive)	W16x45	n=219		Total	0.0	-26.2	0.0	0	0	-697	0.366	D3
Bracing conn	S-4	BR	-	Gravity	-7.4	-0.1	0.0	0	0	0		
at base		108		H. Seismic	40.0	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	2.2	0.0	0.0	0	0	0		
(counteract)		n=105		Total	34.8	-0.1	0.0	0	0	0	0.44	D4
Beam at	S-2, S-4	J	50.0	Gravity	0.0	-2.1	0.0	0	0	-77		
Jnct. level	W14x34	527		H. Seismic	0.0	-19.2	0.0	0	0	-489		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
(additive)	W16x45	n=523		Total	0.0	-21.9	0.0	0	0	-589	0.31	D5
Bracing btwn	S-4	BR	120.0	Gravity	-9.1	0.0	0.0	0	0	0		
Jnct & Wlkwy		508		H. Seismic	-44.6	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.7	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		Total	-56.4	-0.1	0.0	0	0	0	1.69	D6
Beams at	S-2, S-3	TS	80.6	Gravity	0.8	-1.1	0.0	0	-3	132		
Base of	W8x35,	830		H. Seismic	10.3	-5.5	0.4	1	-34	442		
mullions	W10x26			0.300 Gravity	0.2	-0.3	0.0	0	-1	40		
(additive)	W16x77	n=824		Total	11.3	-7.0	0.5	1	-39	613	0.79	D7
Corner mullion	S-3	MUL	40.8	Gravity	-6.5	-0.1	0.7	-2	24	-7		
connections		805		H. Seismic	13.6	-2.1	13.0	-54	490	-130		
at base				0.300 Gravity	2.0	0.0	0.2	-1	7	-2		
(counteract)		n=825		Total	9.0	-2.2	14.0	-57	522	-139	2.12	D8
Mullions at	S-5	MUL	40.8	Gravity	-6.5	-0.1	0.7	-2	24	-7		
top & btm of		805		H. Seismic	-13.6	-2.1	13.0	-54	490	-130		
window	TS			0.300 Gravity	-2.0	0.0	0.2	-1	7	-2		
(additive)	8x4x1/2	n=825		Total	-22.1	-2.2	14.0	-57	522	-139	0.92	D9
Mullions	S-5	MUL	94.6	Gravity	-4.7	-0.4	0.0	0	-2	-16		
within		1006		H. Seismic	-4.9	-5.2	-5.9	6	-294	-235		
window span	TS			0.300 Gravity	-1.4	-0.1	0.0	0	-1	-5		
(additive)	8x4x1/2	n=1012		Total	-11.0	-5.7	-5.9	6	-297	-256	0.72	D10
Tubes at	S-5	BW	57.6	Gravity	-0.8	-0.7	0.0	0	0	-18		
base of		1011		H. Seismic	-2.0	-8.5	-0.9	-20	-47	-476		
window	TS			0.300 Gravity	-0.2	-0.2	0.0	0	0	-5		
(additive)	7x7 3/16	n=1012		Total	-3.1	-9.4	-0.9	-21	-47	-499	1.08	D11
Tubes at	S-5	TW	72.4	Gravity	0.8	0.3	0.0	0	1	-5		
top of		1109		H. Seismic	3.0	4.7	-0.6	13	34	-318		
window	TS			0.300 Gravity	0.2	0.1	0.0	0	0	-2		
(additive)	7x7 3/16	n=1108		Total	4.0	5.1	-0.6	13	35	-325	0.70	D12

Table 50. Forces, Moments, and DCRs (SLO7, no windows, θ_L = 0°, S_{DS} = 1.00g).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Bracing at	S-4	BR	149.1	Gravity	-7.4	-0.1	0.0	0	0	0		
Base		108		H. Seismic	-52.5	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.2	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=105		Total	-62.2	-0.1	0.0	0	0	0	2.58	D1
Column at	S-4	C	118.5	Gravity	-53.3	0.0	0.0	0	0	-1		
Base		104		H. Seismic	-96.1	-0.3	0.4	0	-42	-33		
				0.300 Gravity	-16.0	0.0	0.0	0	0	0		
(additive)	W8x31	n=104		Total	-165.4	-0.3	0.4	0	-42	-34	0.68	D2
Beam at	S-2, S-4	I	50.0	Gravity	0.0	-0.8	0.0	0	0	-56		
Intermediate	W14x34	223		H. Seismic	0.0	-33.0	0.0	0	0	-824		
Levels	W16x40			0.300 Gravity	0.0	-0.2	0.0	0	0	-17		
(additive)	W16x45	n=219		Total	0.0	-34.0	0.0	0	0	-897	0.47	D3
Bracing conn	S-4	BR	-	Gravity	-7.4	-0.1	0.0	0	0	0		
at base		108		H. Seismic	52.5	0.0	0.0	0	0	0		
- Tension				-0.300 Gravity	2.2	0.0	0.0	0	0	0		
(counteract)		n=105		Total	47.3	-0.1	0.0	0	0	0	0.60	D4
Beam at	S-2, S-4	J	50.0	Gravity	0.0	-2.1	0.0	0	0	-77		
Jnct. level	W14x34	527		H. Seismic	0.0	-25.6	0.0	0	0	-653		
(El. 29'-8")	W16x40			0.300 Gravity	0.0	-0.6	0.0	0	0	-23		
(additive)	W16x45	n=523		Total	0.0	-28.4	0.0	0	0	-754	0.40	D5
Bracing btwn	S-4	BR	120.0	Gravity	-9.1	0.0	0.0	0	0	0		
Jnct & Wlkwy		508		H. Seismic	-59.5	0.0	0.0	0	0	0		
- Buckling				0.300 Gravity	-2.7	0.0	0.0	0	0	0		
(additive)	L6x6x1/2	n=526		Total	-71.3	-0.1	0.0	0	0	0	2.33	D6
Beams at	S-2, S-3	TS	43.6	Gravity	-3.4	-7.3	-0.3	0	-13	-65		
Base of	W8x35,	833		H. Seismic	-7.5	-40.5	-5.2	-5	-237	-1263		
mullions	W10x26			0.300 Gravity	-1.0	-2.2	-0.1	0	-4	-19		
(additive)	W16x77	n=827		Total	-11.9	-50.0	-5.5	-5	-253	-1347	0.43	D7
Corner mullion	S-3	MUL	40.8	Gravity	-11.5	-0.4	-0.1	0	-3	-1		
connections		806		H. Seismic	6.5	-1.1	-18.2	-17	-728	-10		
at base				0.300 Gravity	3.4	-0.1	0.0	0	-1	0		
(counteract)		n=828		Total	-1.5	-1.6	-18.3	-17	-731	-11	2.49	D8
Mullions at	S-5	MUL	40.8	Gravity	-11.5	-0.4	-0.1	0	-3	-1		
top & btm of		806		H. Seismic	-6.5	-1.1	-18.2	-17	-728	-10		
window	TS			0.300 Gravity	-3.4	-0.1	0.0	0	-1	0		
(additive)	8x4x1/2	n=828		Total	-21.4	-1.6	-18.3	-17	-731	-11	1.11	D9
Mullions	S-5	MUL	94.6	Gravity	-4.7	-0.4	0.0	0	-2	-16		
within		1006		H. Seismic	-2.1	-2.2	-8.0	5	-400	-103		
window span	TS			0.300 Gravity	-1.4	-0.1	0.0	0	-1	-5		
(additive)	8x4x1/2	n=1012		Total	-8.2	-2.7	-8.0	5	-402	-124	0.75	D10
Tubes at	S-5	BW	57.6	Gravity	-0.8	-0.4	0.0	0	1	-3		
base of		1010		H. Seismic	-2.2	-10.0	0.7	26	36	-577		
window	TS			0.300 Gravity	-0.2	-0.1	0.0	0	0	-1		
(additive)	7x7 3/16	n=1009		Total	-3.2	-10.6	0.7	27	36	-581	1.22	D11
Tubes at	S-5	TW	72.4	Gravity	0.8	0.4	0.0	0	-1	-12		
top of		1112		H. Seismic	2.1	5.6	0.7	-18	-44	-362		
window	TS			0.300 Gravity	0.2	0.1	0.0	0	0	-4		
(additive)	7x7 3/16	n=1112		Total	3.1	6.1	0.7	-18	-45	-377	0.82	D12

Table 51. Mullion Connections at Their Bases (SLO7, no windows, $\theta_L = 0°$, $S_{DS} = 1.00g$).

Component	Drawing/ Section #	Member ID/End	Length (in.)	Load Type	P (kips)	V2,Vy (kips)	V3,Vx (kips)	T (k-in)	M2,My (k-in)	M3,Mx (k-in)	DCR	App #
Corner mullion	S-3	MUL801	-	Gravity	-10.7	-0.5	-0.2	0	-5	-1		
connections				H. Seismic	6.6	-1.6	-16.6	35	-654	-7		
at base				0.300 Gravity	3.2	-0.1	-0.1	0	-2	0		
(counteract)		n=804		Total	-0.9	-2.2	-16.9	35	-661	-7	2.25	D8
Corner mullion	S-3	MUL802	-	Gravity	-6.8	-0.1	0.7	-1	28	-7		
connections				H. Seismic	16.5	-2.6	7.9	-23	367	-101		
at base				0.300 Gravity	2.0	0.0	0.2	0	8	-2		
(counteract)		n=808		Total	11.8	-2.7	8.8	-25	403	-111	1.68	D8
Corner mullion	S-3	MUL803	-	Gravity	-5.7	-0.1	-0.6	1	-20	-5		
connections				H. Seismic	16.7	-1.9	-9.0	32	-364	-99		
at base				0.300 Gravity	1.7	0.0	-0.2	0	-6	-2		
(counteract)		n=809		Total	12.7	-2.1	-9.7	34	-390	-106	1.63	D8
Corner mullion	S-3	MUL804	-	Gravity	-6.6	-0.1	-0.6	0	-23	-9		
connections				H. Seismic	17.3	-1.3	-7.3	17	-363	-98		
at base				0.300 Gravity	2.0	0.0	-0.2	0	-7	-3		
(counteract)		n=824		Total	12.7	-1.5	-8.0	17	-392	-109	1.65	D8
Corner mullion	S-3	MUL805	-	Gravity	-6.5	-0.1	0.7	-2	24	-7		
connections				H. Seismic	16.5	-1.0	11.8	-40	477	-129		
at base				0.300 Gravity	2.0	0.0	0.2	-1	7	-2		
		n=825		Total	11.9	-1.2	12.7	-42	509	-138	2.10	D8
Corner mullion	S-3	MUL806	-	Gravity	-11.5	-0.4	-0.1	0	-3	-1		
connections				H. Seismic	6.5	-1.1	-18.2	17	-728	-10		
at base				0.300 Gravity	3.4	-0.1	0.0	0	-1	0		
(counteract)		n=828		Total	-1.5	-1.6	-18.3	17	-731	-11	2.49	D8
									Average DCR =		1.97	

7 Summary

This report presents the detailed seismic evaluation of the FAA ATCTs at the locations given in Table 1. These include Type L towers located in Salinas, San Carlos, and Palo Alto, CA. A unique eccentrically braced steel frame tower located in San Luis Obispo was also evaluated. Each of these towers was evaluated based on the maximum considered earthquake defined by 1997 NEHRP Recommended Provisions (FEMA 302). Both Type L and the San Luis Obispo towers were evaluated based on several directions of loading and an extreme assumption that the cab windows will not fail and will work as fully effective shear walls.

Type L ATCTs

The San Carlos ATCT is the most critical Type L tower, due to excessive deflections in the tower cab. These deflections were due to large rotations of supporting members at the shaft roof. The cab columns (corner mullions) connection base plates were also overstressed as indicated by very large DCRs. Hinges would form at the base of each mullion due to base plate bending failure, causing a collapse mechanism at very low seismic motions.

An upgrade approach was developed and demonstrated that reduces deflections to acceptable levels and protects the vulnerable connections. This upgrade consists of welding deep structural tubing members to the base of each corner mullion in a pentagon configuration as shown in Figure 11. The mullions themselves were also stiffened and strengthened by welding 5 in. x 1.5 in. plates on both faces of the mullions.

San Luis Obispo ATCT

The shaft braces were the most critically stressed components in the San Luis Obispo shafts. These would buckle at several floor levels. However, when the braces were assumed to be tension only members, they had adequate capacity. There could be slight yielding of these braces, but this would be very limited and deflections would be kept within acceptable levels.

Deflection in the tower cab could be large, but within acceptable levels. The most vulnerable cab component is the connection of the corner mullions to their base plates. This is due to shear failure of the fillet welds at this connection. However, serious damage to these connections should be prevented by redistribution of forces to other mullions and other building components. Therefore, the San Luis Obispo tower passed this evaluation by meeting the life-safety requirements.

APPENDIX A

Member Properties and Loads for Salinas Type L ATCT

Member #	Description	SAP2000 Designation	Member Length (in.)	Weight per Length (k/in)	Mass per Length (k-s2/in2)
floor 2					
1	16B26	W1626AB	130	0.02537	0.00006565
2	16B26	W16X26A	110	0.02537	0.00006565
3	W16 x 40	W16X40A	84	0.02660	0.00006884
4	W16 x 40	W16X40A	81	0.02660	0.00006884
5	W16 x 40	W16X40A	75	0.02660	0.00006884
6	16B26	W16X26B	239	0.04162	0.00010772
7	W16 x 40	W16X40B	75	0.04039	0.00010453
8	W16 x 40	W16X40B	81	0.04039	0.00010453
9	W16 x 40	W1640BB	84	0.04039	0.00010453
10	16B26	W1626CB	136	0.02776	0.00007185
11	16B26	W16X26C	116	0.02776	0.00007185
12	16B26	W16X26C	252	0.02776	0.00007185
13	8C11.5	C8X11.5A	90	0.02776	0.00007185
floor 3					
14	16B26	W1626DB	130	0.02525	0.00006536
15	16B26	W16X26D	110	0.02525	0.00006536
16	W16 x 40	W16X40C	84	0.02648	0.00006853
17	W16 x 40	W16X40C	81	0.02648	0.00006853
18	W16 x 40	W16X40C	75	0.02648	0.00006853
19	16B26	W16X26E	239	0.04143	0.00010723
20	W16 x 40	W16X40D	75	0.04021	0.00010406
21	W16 x 40	W16X40D	81	0.04021	0.00010406
22	W16 x 40	W1640DB	84	0.04021	0.00010406
23	16B26	W1626FB	136	0.02764	0.00007153
24	16B26	W16X26F	116	0.02764	0.00007153
25	16B26	W16X26F	252	0.02764	0.00007153
26	8C11.5	C8X11.5B	90	0.02764	0.00007153
floor 4					
27	16B26	W1626GB	120	0.02553	0.00006608
28	16B26	W16X26G	120	0.02553	0.00006608
29	W16 x 40	W16X40E	84	0.02684	0.00006946
30	W16 x 40	W16X40E	81	0.02684	0.00006946
31	W16 x 40	W16X40E	75	0.02684	0.00006946
32	16B26	W16X26H	239	0.04280	0.00011077
33	W16 x 40	W16X40F	75	0.04149	0.00010739
34	W16 x 40	W16X40F	81	0.04149	0.00010739
35	W16 x 40	W1640FB	84	0.04149	0.00010739
36	16B26	W1626IB	126	0.02950	0.00007634
37	16B26	W16X26I	126	0.02950	0.00007634
38	16B26	W16X26I	252	0.02950	0.00007634
39	8C11.5	C8X11.5C	90	0.02950	0.00007634
top of bent					
40	W16 x 40	W16X40G	23.57	0.01779	0.00004605
41	W16 x 40	W16X40G	69.00	0.01779	0.00004605
42	W16 x 40	W16X40G	27.00	0.01779	0.00004605
43	W16 x 40	W16X40H	96.00	0.01247	0.00003226
44	W16 x 40	W16X40H	23.57	0.01247	0.00003226
45	16 WF 71	W16X67A	119.57	0.01280	0.00003312
46	16 WF 71	W16X67B	47.10	0.01813	0.00004691
47	16 WF 71	W16X67B	72.47	0.01813	0.00004691
48	16 WF 88	W16X89A	119.57	0.01771	0.00004583
49	16 WF 88	W16X89B	119.57	0.02304	0.00005962
50	16 WF 71	W16X67C	72.47	0.02345	0.00006069
51	16 WF 71	W16X67C	47.10	0.02345	0.00006069
52	16 WF 71	W16X67D	119.57	0.01813	0.00004691
53	10 W 29	W10X26	80.00	0.00025	0.00000065

54	10 W 29	W10X26	59.10	0.00025	0.00000065
55	10 W 29	W10X26	20.90	0.00025	0.00000065
56	10 W 29	W10X26	40.00	0.00025	0.00000065
57	10 W 29	W10X26	40.00	0.00025	0.00000065
58	10 W 29	W10X26	20.90	0.00025	0.00000065
59	10 W 29	W10X26	59.10	0.00025	0.00000065
60	W 10 x 49	W10X49	74.77	0.00000	0.00000000
61	W 10 x 49	W10X49	74.77	0.00000	0.00000000
62	10 W 72	W10X68	30.80	0.00033	0.00000086
63	10 W 72	W10X68	90.44	0.00033	0.00000086
64	10 W 72	W10X68	43.40	0.00033	0.00000086
65	10 W 72	W10X68	86.00	0.00033	0.00000086
66	10 W 72	W10X68	30.80	0.00033	0.00000086
67	10 W 72	W10X68	90.44	0.00033	0.00000086
68	Concrete Bent	TBNT	92.91	0.01341	0.00003471
69	Concrete Bent	TBNT	28.71	0.01341	0.00003471
70	Concrete Bent	TBNT	56.57	0.01341	0.00003471
71	Concrete Bent	TBNT	92.91	0.01341	0.00003471
72	Concrete Bent	TBNT	28.71	0.01341	0.00003471
73	Concrete Bent	TBNT	56.57	0.01341	0.00003471
74	Concrete Bent	TBNT	121.62	0.01341	0.00003471
75	Concrete Bent	TBNT	56.57	0.01341	0.00003471
76	Concrete Bent	TBNT	121.62	0.01341	0.00003471
77	Concrete Bent	TBNT	56.57	0.01341	0.00003471
cab floor					
78	12 W 27	W12X26A	69.00	0.01296	0.00003354
79	12 W 27	W12X26B	62.99	0.00725	0.00001876
80	12 W 27	W12X26B	7.34	0.00725	0.00001876
81	12 W 27	W12X26B	77.28	0.00725	0.00001876
82	12 W 27	W12X26B	44.37	0.00725	0.00001876
83	12 W 27	W12X26C	121.49	0.01683	0.00004357
84	12 W 27	W12X26C	70.45	0.01683	0.00004357
85	12 W 27	W12X26D	95.97	0.02255	0.00005835
86	12 W 27	W12X26D	95.97	0.02255	0.00005835
87	12 W 27	W12X26D	21.03	0.02255	0.00005835
88	12 W 27	W12X26D	23.34	0.02255	0.00005835
89	12 W 27	W12X26A	51.64	0.01296	0.00003354
90	12 W 27	W12X26A	74.95	0.01296	0.00003354
91	12 W 27	W12X26A	21.03	0.01296	0.00003354
92	C 6 x 8.2	C6X8.2A	44.50	0.00979	0.00002534
93	12 B 19	W12X19	79.38	0.00979	0.00002534
94	12 B 19	W12X19	99.77	0.00979	0.00002534
95	12 B 19	W12X19	44.81	0.00979	0.00002534
96	C 6 x 8.2	C6X8.2A	50.03	0.00979	0.00002534
97	12 B 19	W12X19	111.98	0.00979	0.00002534
98	12 B 19	W12X19	111.98	0.00979	0.00002534
99	12 B 19	W12X19	71.41	0.00979	0.00002534
100	12 B 19	W12X19	111.98	0.00979	0.00002534
101	12 B 19	W12X19	111.98	0.00979	0.00002534
102	12 W 27	W12X26E	23.43	0.00987	0.00002555
103	12 W 27	W12X26E	74.95	0.00987	0.00002555
104	12 W 27	W12X26E	74.98	0.00987	0.00002555
105	12 W 27	W12X26E	23.34	0.00987	0.00002555
bot. of glass					
106	C 4 x 7.25	C47.25B	96.00	0.00134	0.00000346
107	C 4 x 7.25	C47.25B	96.00	0.00134	0.00000346
108	C 4 x 7.25	C47.25B	95.99	0.00134	0.00000346
109	C 4 x 7.25	C47.25B	42.87	0.00134	0.00000346
110	C 4 x 7.25	C47.25B	53.13	0.00134	0.00000346
111	C 4 x 7.25	C47.25B	95.97	0.00134	0.00000346
112	C 4 x 7.25	C47.25B	95.97	0.00134	0.00000346
113	C 4 x 7.25	C47.25B	95.97	0.00134	0.00000346
114	C 4 x 7.25	C47.25B	95.97	0.00134	0.00000346
115	C 4 x 7.25	C47.25B	53.13	0.00134	0.00000346
116	C 4 x 7.25	C47.25B	42.87	0.00134	0.00000346

117	C 4 x 7.25	C47.25B	95.99	0.00134	0.00000346
top of glass					
118	C 6 x 8.2	C6X8.2B	81.00	0.00071	0.00000185
119	C 6 x 8.2	C6X8.2B	37.40	0.00071	0.00000185
120	C 6 x 8.2	C6X8.2B	37.40	0.00071	0.00000185
121	C 6 x 8.2	C6X8.2B	81.00	0.00071	0.00000185
122	C 6 x 8.2	C6X8.2B	80.96	0.00071	0.00000185
123	C 6 x 8.2	C6X8.2B	37.43	0.00071	0.00000185
124	C 6 x 8.2	C6X8.2B	37.43	0.00071	0.00000185
125	C 6 x 8.2	C6X8.2B	15.54	0.00071	0.00000185
126	C 6 x 8.2	C6X8.2B	65.42	0.00071	0.00000185
127	C 6 x 8.2	C6X8.2B	80.97	0.00071	0.00000185
128	C 6 x 8.2	C6X8.2B	37.35	0.00071	0.00000185
129	C 6 x 8.2	C6X8.2B	37.35	0.00071	0.00000185
130	C 6 x 8.2	C6X8.2B	80.97	0.00071	0.00000185
131	C 6 x 8.2	C6X8.2B	80.97	0.00071	0.00000185
132	C 6 x 8.2	C6X8.2B	37.35	0.00071	0.00000185
133	C 6 x 8.2	C6X8.2B	37.35	0.00071	0.00000185
134	C 6 x 8.2	C6X8.2B	80.97	0.00071	0.00000185
135	C 6 x 8.2	C6X8.2B	65.42	0.00071	0.00000185
136	C 6 x 8.2	C6X8.2B	15.54	0.00071	0.00000185
137	C 6 x 8.2	C6X8.2B	37.43	0.00071	0.00000185
138	C 6 x 8.2	C6X8.2B	37.43	0.00071	0.00000185
139	C 6 x 8.2	C6X8.2B	80.96	0.00071	0.00000185
cab roof					
140	W 14 x 30	W14X30A	6.00	0.00801	0.00002072
141	W 14 x 30	W14X30A	78.00	0.00801	0.00002072
142	W 14 x 30	W14X30A	38.00	0.00801	0.00002072
143	W 14 x 30	W14X30B	38.00	0.00417	0.00001079
144	W 14 x 30	W14X30B	78.00	0.00417	0.00001079
145	W 14 x 30	W14X30B	6.00	0.00417	0.00001079
146	14 B 22	W14X22A	84.56	0.00417	0.00001079
147	14 B 22	W14X22A	74.85	0.00417	0.00001079
148	14 B 22	W14X22A	11.48	0.00417	0.00001079
149	14 B 22	W14X22B	73.15	0.00971	0.00002514
150	W 14 x 30	W14X30C	84.61	0.00971	0.00002514
151	W 14 x 30	W14X30C	10.46	0.00971	0.00002514
152	W 14 x 30	W14X30C	64.23	0.00971	0.00002514
153	W 14 x 30	W14X30C	37.72	0.00971	0.00002514
154	W 14 x 30	W14X30C	46.97	0.00971	0.00002514
155	W 14 x 30	W14X30D	46.97	0.01355	0.00003507
156	W 14 x 30	W14X30D	37.72	0.01355	0.00003507
157	W 14 x 30	W14X30D	64.23	0.01355	0.00003507
158	W 14 x 30	W14X30D	10.46	0.01355	0.00003507
159	W 14 x 30	W14X30D	84.61	0.01355	0.00003507
160	14 B 22	W14X22C	73.15	0.01355	0.00003507
161	14 B 22	W14X22D	11.48	0.00801	0.00002072
162	14 B 22	W14X22D	74.85	0.00801	0.00002072
163	14 B 22	W14X22D	84.56	0.00801	0.00002072
164	14 B 22	W14X22E	144.00	0.00625	0.00001618
165	14 B 22	W14X22E	144.00	0.00625	0.00001618
166	14B26	W14X26	144.00	0.00625	0.00001618
167	14B26	W14X26	144.00	0.00625	0.00001618
168	14 B 22	W14X22E	59.90	0.00625	0.00001618
169	14B26	W14X26	144.00	0.00625	0.00001618
170	14B26	W14X26	144.00	0.00625	0.00001618
171	14 B 22	W14X22E	59.90	0.00625	0.00001618
172	14 B 22	W14X22E	144.00	0.00625	0.00001618
173	14 B 22	W14X22E	144.00	0.00625	0.00001618
174	W 14 x 30	W14X30E	4.50	0.00625	0.00001618
175	W 14 x 30	W14X30E	78.00	0.00625	0.00001618
176	W 14 x 30	W14X30E	76.00	0.00625	0.00001618
177	W 14 x 30	W14X30E	78.00	0.00625	0.00001618
178	W 14 x 30	W14X30E	4.50	0.00625	0.00001618

2nd Floor

Tower Stairs:

Total Stair Weight =	2828	lb.
Mechanical & Electrical:		
w =	5	lb./sq. ft.
Area =	354.74	sq. ft.
Weight =	1774	lb.

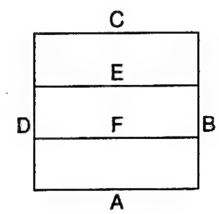
2nd Floor Summary

Eccentrically Distributed Loads:

- Weight from additional members =	0	lb.
- Concrete Deck =	22171	lb.
- Interior Walls = (including doors etc.)	2662	lb. (1/3 floor 1 + 2/3 floor 2)
- Stairs (lb) =	2828	lb. (18 treads + landing)
- Mech & Elect (lb) =	1774	lb.
- Misc (lb) =	3547	lb.
Sub-total (lb):	32982	lb.
Distribution to exterior beams:	16491	lb.
Distribution to interior beams:	16491	lb.
exterior beam length:	79.71	ft.
Total length of interior beams:	49.50	ft.

Uniformly Distributed Loads:

Wt. of exterior walls: (including doors etc.)	15549	lb. (1/3 floor 1 + 2/3 floor 2)



Layout - Plan View

Wall A = 0.5286   Wall B = 0.6000
Wall C = 1.4714   Wall D = 1.4000

Exterior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Location
1	W1626AB	129.57	304.42	0.025368	A
2	W16X26A	109.57	304.42	0.025368	A
3	W16X40A	83.57	319.19	0.026599	B
4	W16X40A	81.00	319.19	0.026599	B
5	W16X40A	74.57	319.19	0.026599	B
6	W16X26B	239.14	499.46	0.041622	C
7	W16X40B	74.57	484.69	0.040391	D
8	W16X40B	81.00	484.69	0.040391	D
9	W1640BB	83.57	484.69	0.040391	D

Interior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Location
10	W1626CB	136.00	333.15	0.027762	F
11	W16X26C	116.00	333.15	0.027762	F
12	W16X26C	252.00	333.15	0.027762	E
13	C8X11.5A	90.00	333.15	0.027762	F-A

Total Self Weight of Beams:	3808.8	lb.
Total Additional Weight to Beams:	48.5	lb.

3rd Floor

Tower Stairs:

Total Stair Weight =	2828	lb.
Mechanical & Electrical:		
w =	5	lb./sq. ft.
Area =	354.74	sq. ft.
Weight =	1774	lb.

Misc. (equipment, connections, etc.)

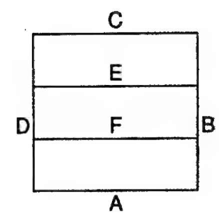
w =	10	lb./sq. ft.
Total Floor Area =	354.74	sq. ft.
Weight =	3547	lb.

3rd Floor Summary

Eccentrically Distributed Loads:

- Weight from additional members =	0	lb.
- Concrete Deck =	22171	lb.
- Interior Walls (including doors etc.) =	2515	lb. (1/3 floor below + 2/3 floor above)
- Stairs (lb) =	2828	lb.
- Mech & Elect (lb) =	1774	lb.
- Misc (lb) =	3547	lb.
Sub-total (lb):	32835	lb.

Distribution to exterior beams:	16418	lb.
Distribution to interior beams:	16418	lb.
exterior beam length:	79.71	ft.
interior beam length:	49.50	ft.



Layout - Plan View

Uniformly Distributed Loads:

Wt. of exterior walls: (including doors etc.)	15478	lb. (1/3 floor below + 2/3 floor above)

Wall A = 0.5286 Wall B = 0.6000
Wall C = 1.4714 Wall D = 1.4000

Exterior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Location
14	W1626DB	129.57	303.04	0.025253	A
15	W16X26D	109.57	303.04	0.025253	A
16	W16X40C	83.57	317.75	0.026479	B
17	W16X40C	81.00	317.75	0.026479	B
18	W16X40C	74.57	317.75	0.026479	B
19	W16X26E	239.14	497.22	0.041435	C
20	W16X40D	74.57	482.51	0.040209	D
21	W16X40D	81.00	482.51	0.040209	D
22	W1640DB	83.57	482.51	0.040209	D

Interior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Location
23	W1626FB	136.00	331.67	0.027639	F
24	W16X26F	116.00	331.67	0.027639	F
25	W16X26F	252.00	331.67	0.027639	E
26	C8X11.5B	90.00	331.67	0.027639	F-A

Total Self Weight of Beams:	3808.8	lb.
Total Additional Weight to Beams:	48.3	lb.

4th Floor

Tower Stairs:

Total Stair Weight =	2761	lb.

Mechanical & Electrical:

w =	5	lb./sq. ft.
Area =	367.32	sq. ft.
Weight =	1837	lb.

Misc. (equipment, connections, etc.)

w =	10	lb./sq. ft.

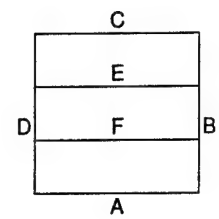
Total Floor Area =	367.32	sq. ft.
Weight =	3673	lb

4th Floor Summary

Eccentrically Distributed Loads:

- Weight from additional members =	0	lb.
- Concrete Deck =	23325	lb.
- Interior Walls = (including doors etc.)	3447	lb.
- Stairs (lb) =	2761	lb.
- Mech & Elect (lb) =	1837	lb.
- Misc (lb) =	3673	lb.
Sub-total (lb):	35042	lb.

Distribution to exterior beams:	17521	lb.
Distribution to interior beams:	17521	lb.
exterior beam length:	79.71	ft.
interior beam length:	49.50	ft.



Layout - Plan View

Uniformly Distributed Loads:

Wt. Of exterior walls: 15162 lb.
(including doors etc.)

Wall A = 0.5286 Wall B = 0.6000
Wall C = 1.4714 Wall D = 1.4000

Exterior beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Location
27	W1626GB	119.57	306.39	0.025532	A
28	W16X26G	119.57	306.39	0.025532	A
29	W16X40E	83.57	322.08	0.026840	B
30	W16X40E	81.00	322.08	0.026840	B
31	W16X40E	74.57	322.08	0.026840	B
32	W16X26H	239.14	513.62	0.042802	C
33	W16X40F	74.57	497.93	0.041494	D
34	W16X40F	81.00	497.93	0.041494	D
35	W1640FB	83.57	497.93	0.041494	D

Interior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Location
36	W1626IB	126.00	353.96	0.029497	F
37	W16X26I	126.00	353.96	0.029497	F
38	W16X26I	252.00	353.96	0.029497	E
39	C8X11.5C	90.00	353.96	0.029497	F-A

Total Self Weight of Beams:	3808.8	lb.
Total Additional Weight to Beams:	50.2	lb.

Top of Bent

Tower Stairs:

Total Stair Weight = 2096 lb.

Mechanical & Electrical:

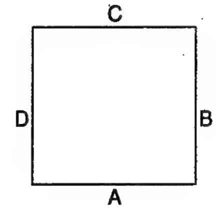
w =	5	lb./sq. ft.
Area =	440.00	sq. ft.
Weight =	2200	lb.

Misc. (equipment, connections, etc.)

w =	0	lb./sq. ft.
Total Floor Area =	440.00	sq. ft.
Weight =	0	lb

Top of Bent Summary

Eccentrically Distributed Loads:		
Weight from additional members:	4381	lb.
all loads except ext. walls:	10443	lb.
Stairs :	2096	lb.
- Mech & Elect (lb) =	2200	lb.
- Misc (lb) =	0	lb.
Sub-total (lb):	19120	lb.
Distribution to exterior beams:	9560	lb.
Distribution to interior beams:	9560	lb.
exterior beam length:	79.71	ft.
total length of interior beams:	59.40	ft.
Uniformly Distributed Loads:		
Wt. Of exterior walls:	7459	lb. (1/3 floor 4 + 2/3 between bent and cab floor)
(including doors etc.)		



Layout - Plan View

Dist AB = 1.0000 Dist AD = 1.5330
Dist CD = 1.0000 Dist BC = 0.4670

Exterior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
40	W16X40G	23.57	213.50	0.017792
41	W16X40G	69.00	213.50	0.017792
42	W16X40G	27.00	213.50	0.017792
43	W16X40H	96.00	149.58	0.012465
44	W16X40H	23.57	149.58	0.012465
45	W16X67A	119.57	149.58	0.012465
46	W16X67B	47.10	213.50	0.017792
47	W16X67B	72.47	213.50	0.017792
48	W16X89A	119.57	213.50	0.017792
49	W16X89B	119.57	277.43	0.023119
50	W16X67C	72.47	277.43	0.023119
51	W16X67C	47.10	277.43	0.023119
52	W16X67D	119.57	213.50	0.017792

Interior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
53	W10X26	80.00	0.00	0
54	W10X26	59.10	0.00	0
55	W10X26	20.90	0.00	0
56	W10X26	40.00	0.00	0
57	W10X26	40.00	0.00	0
58	W10X26	20.90	0.00	0
59	W10X26	59.10	0.00	0
60	W10X49	74.77	0.00	0
61	W10X49	74.77	0.00	0
62	W10X68	30.80	0.00	0
63	W10X68	90.44	0.00	0
64	W10X68	43.40	0.00	0
65	W10X68	86.00	0.00	0
66	W10X68	30.80	0.00	0
67	W10X68	90.44	0.00	0
68	TBNT	92.91	160.95	0.013412
69	TBNT	28.71	160.95	0.013412
70	TBNT	56.57	160.95	0.013412
71	TBNT	92.91	160.95	0.013412
72	TBNT	28.71	160.95	0.013412
73	TBNT	56.57	160.95	0.013412
74	TBNT	121.62	160.95	0.013412
75	TBNT	56.57	160.95	0.013412
76	TBNT	121.62	160.95	0.013412

77	TBNT	56.57	160.95	0.013412

Total Self Weight of Beams:	33328.5	lb.
Total Additional Weight to Beams:	26.6	lb.

Cab Floor

Stairs:

Total stair weight =	133	lb.

Mechanical & Electrical:

w =	5	lb./sq. ft.
Area =	400.00	sq. ft.
Weight =	2000	lb.

Misc. (equipment, connections, etc.)

w =	0	lb./sq. ft.
Total Floor Area =	400.00	sq. ft.
Weight =	0	lb

Cab Floor Summary

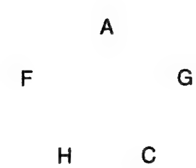
Eccentrically Distributed Loads:

- Weight from additional members :	0	lb.
All loads except ext. walls:	18125	lb.
-Stairs:	133	lb.
- Mech & Elect (lb) =	2000	lb.
- Misc (lb) =	0	lb.
Sub-total (lb):	20258	lb.

Distribution to exterior beams:	10129	lb.
Distribution to interior beams:	10129	lb.
total length of exterior beams:	69.74	ft.
total length of interior beams:	86.21	ft.

Uniformly Distributed Loads:

Wt. Of exterior walls: (including doors etc.)	2268	lb.



Layout - Plan View

Wall AF = 1.6320 Wall AG = 1.1600
Wall CG = 0.3680 Wall FH = 0.8400

Exterior beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
78	W12X26A	69.00	154.53	0.012877
79	W12X26B	62.99	85.98	0.007165
80	W12X26B	7.34	85.98	0.007165
81	W12X26B	77.28	85.98	0.007165
82	W12X26B	44.37	85.98	0.007165
83	W12X26C	121.49	201.01	0.016750
84	W12X26C	70.45	201.01	0.016750
85	W12X26D	95.97	269.56	0.022463
86	W12X26D	95.97	269.56	0.022463
87	W12X26D	21.03	269.56	0.022463
88	W12X26D	23.34	269.56	0.022463
89	W12X26A	51.64	154.53	0.012877
90	W12X26A	74.95	154.53	0.012877
91	W12X26A	21.03	154.53	0.012877

Interior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
92	C6X8.2A	44.50	117.49	0.009791
93	W12X19	79.38	117.49	0.009791
94	W12X19	99.77	117.49	0.009791
95	W12X19	44.81	117.49	0.009791

96	C6X8.2A	50.03	117.49	0.009791
97	W12X19	111.98	117.49	0.009791
98	W12X19	111.98	117.49	0.009791
99	W12X19	71.41	117.49	0.009791
100	W12X19	111.98	117.49	0.009791
101	W12X19	111.98	117.49	0.009791
102	W12X26E	23.43	117.49	0.009791
103	W12X26E	74.95	117.49	0.009791
104	W12X26E	74.98	117.49	0.009791
105	W12X26E	23.34	117.49	0.009791

Total Self Weight of Beams: 3481 lb.
Total Additional Weight to Beams: 22.8 lb.

Bottom of Glass

Eccentrically Distributed Loads:

Weight from additional members:	0	lb.
wt. from glass:	681	lb.
wt. from other:	0	lb.
Sub-total (lb):	681	lb.

Distribution to exterior beams: 681 lb. (no interior beams)
exterior beam length: 79.99 ft.

Uniformly Distributed Loads:
Wt. Of exterior walls: 600 lb.
(including doors etc.)

A
F G
H C
Layout - Plan View

Dist AF = 1.0000 Dist AG = 1.0000
Dist GH = 1.0000 Dist FH = 1.0000

Exterior beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
106	C47.25B	96.00	16.02	0.001335
107	C47.25B	96.00	16.02	0.001335
108	C47.25B	95.99	16.02	0.001335
109	C47.25B	42.87	16.02	0.001335
110	C47.25B	53.13	16.02	0.001335
111	C47.25B	95.97	16.02	0.001335
112	C47.25B	95.97	16.02	0.001335
113	C47.25B	95.97	16.02	0.001335
114	C47.25B	95.97	16.02	0.001335
115	C47.25B	53.13	16.02	0.001335
116	C47.25B	42.87	16.02	0.001335
117	C47.25B	95.99	16.02	0.001335

Total Self Weight of Beams: 579.9 lb.
Total Additional Weight to Beams: 1.3 lb.

Top of Glass

Eccentrically Distributed Loads:

Weight from additional members:	0	lb.
wt. from glass:	845	lb.
wt. from other:	0	lb.
Sub-total (lb):	845	lb.

Distribution to exterior beams: 845 lb. (no interior beams)
exterior beam length: 98.63 ft.

Uniformly Distributed Loads:
Wt. Of exterior walls: 0 lb.
(including doors etc.)

A
F G
H C

Dist AF = 1.0000 Dist AG = 1.0000 Layout - Plan View
Dist GH = 1.0000 Dist FH = 1.0000

Exterior beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
118	C6X8.2B	81.00	8.56	0.000714
119	C6X8.2B	37.40	8.56	0.000714
120	C6X8.2B	37.40	8.56	0.000714
121	C6X8.2B	81.00	8.56	0.000714
122	C6X8.2B	80.96	8.56	0.000714
123	C6X8.2B	37.43	8.56	0.000714
124	C6X8.2B	37.43	8.56	0.000714
125	C6X8.2B	15.54	8.56	0.000714
126	C6X8.2B	65.42	8.56	0.000714
127	C6X8.2B	80.97	8.56	0.000714
128	C6X8.2B	37.35	8.56	0.000714
129	C6X8.2B	37.35	8.56	0.000714
130	C6X8.2B	80.97	8.56	0.000714
131	C6X8.2B	80.97	8.56	0.000714
132	C6X8.2B	37.35	8.56	0.000714
133	C6X8.2B	37.35	8.56	0.000714
134	C6X8.2B	80.97	8.56	0.000714
135	C6X8.2B	65.42	8.56	0.000714
136	C6X8.2B	15.54	8.56	0.000714
137	C6X8.2B	37.43	8.56	0.000714
138	C6X8.2B	37.43	8.56	0.000714
139	C6X8.2B	80.96	8.56	0.000714

Total Self Weight of Beams:	808.8	lb.
Total Additional Weight to Beams:	0.8	lb.

Cab Roof

Mechanical & Electrical:		
w =	5	lb./sq. ft.
Area =	770.00	sq. ft.
Weight =	3850	lb.
Misc. (equipment, connections, etc.)		
w =	0	lb./sq. ft.
Total Floor Area =	770.00	sq. ft.
Weight =	0	lb

Cab Roof Summary

Eccentrically Distributed Loads:		
Weight from additional members:	0	lb.
Roof weight minus cladding weight:	15071	lb.
Mech & Elect:	3850	lb.
Misc:	0	lb.
Sub-total (lb):	18921	lb.
Distribution to exterior beams:	9461	lb.
Distribution to interior beams:	9461	lb.
total length of exterior beams:	101.67	ft.
total length of interior beams:	126.07	ft.

A
F G
D H C

Layout - Plan View

Uniformly Distributed Loads:		
Wt. Of exterior walls: (including doors etc.)	1350	lb. (from top of glass to roof)

Wall AF = 1.6050 Wall AG = 1.1100
Wall GH = 0.3950 Wall FH = 0.8900

Exterior beams

Member #	Model Section	Member Length	Additional Load	Additional Load

		(in.)	(lb./ft.)	(k/in)
140	W14X30A	6.00	96.09	0.008008
141	W14X30A	78.00	96.09	0.008008
142	W14X30A	38.00	96.09	0.008008
143	W14X30B	38.00	50.03	0.004169
144	W14X30B	78.00	50.03	0.004169
145	W14X30B	6.00	50.03	0.004169
146	W14X22A	84.56	50.03	0.004169
147	W14X22A	74.85	50.03	0.004169
148	W14X22A	11.48	50.03	0.004169
149	W14X22B	73.15	116.56	0.009714
150	W14X30C	84.61	116.56	0.009714
151	W14X30C	10.46	116.56	0.009714
152	W14X30C	64.23	116.56	0.009714
153	W14X30C	37.72	116.56	0.009714
154	W14X30C	46.97	116.56	0.009714
155	W14X30D	46.97	162.62	0.013552
156	W14X30D	37.72	162.62	0.013552
157	W14X30D	64.23	162.62	0.013552
158	W14X30D	10.46	162.62	0.013552
159	W14X30D	84.61	162.62	0.013552
160	W14X22C	73.15	162.62	0.013552
161	W14X22D	11.48	96.09	0.008008
162	W14X22D	74.85	96.09	0.008008
163	W14X22D	84.56	96.09	0.008008

Interior Beams

Member #	Model Section	Member Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
164	W14X22E	144.00	75.04	0.006254
165	W14X22E	144.00	75.04	0.006254
166	W14X26	144.00	75.04	0.006254
167	W14X26	144.00	75.04	0.006254
168	W14X22E	59.90	75.04	0.006254
169	W14X26	144.00	75.04	0.006254
170	W14X26	144.00	75.04	0.006254
171	W14X22E	59.90	75.04	0.006254
172	W14X22E	144.00	75.04	0.006254
173	W14X22E	144.00	75.04	0.006254
174	W14X30E	4.50	75.04	0.006254
175	W14X30E	78.00	75.04	0.006254
176	W14X30E	76.00	75.04	0.006254
177	W14X30E	78.00	75.04	0.006254
178	W14X30E	4.50	75.04	0.006254

Total Self Weight of Beams: 5851.0 lb.
Total Additional Weight to Beams: 20.4 lb.

DOORS: Number	Location	Description	Area (sq. in.)	Total Weight (lb.)
1	FL-1 exit	"B"-self-closing	6468.00	229
2	FL-1 stair	"	6468.00	229
3	FL-1 Telco	"	5439.00	193
4	FL-2 stair	"	6468.00	229
5	FL-all c. chase	solid core	1440.00	58
6	FL-2 stair	"B"-self-closing	6468.00	229
7	omitted	---	---	---
8	FL-4 bath	"B"-self-closing	5439.00	193
9	FL-4 stair	"	6468.00	229
10	FL-4/5 landing	"	5512.00	195
11	FL-4 closet	"	5439.00	193
12	FL-4 closet	"	5439.00	193
13	FL-5 catwalk	"	3251.25	115

DOORS JAMBS:

Door Number	Area (sq. in.)	Weight (lb./ft.)	Top Stud Weight (lb./ft.)	Total Weight (lb.)
1	1.11	3.76	1.77	69
2	1.11	3.76	1.77	69
3	1.11	3.76	1.77	67
4	1.11	3.76	1.77	69
5	1.11	3.76	1.77	44
6	1.11	3.76	1.77	69
7	---	---	---	---
8	1.11	3.76	1.77	67
9	1.11	3.76	1.77	69
10	1.11	3.76	1.77	65
11	1.11	3.76	1.77	67
12	1.11	3.76	1.77	67
13	1.11	3.76	1.77	50

INTERIOR WALLS:

Floor	Wall Number	Wall Area (sq. ft.)	Unit Weight of wall (lb./sq. ft.)	Total wt.--wall material only (lb.)	Total wt. walls + doors (lb.)	
1 (ground)	1	27.79	6.33	176	176	
	2	26.42	6.33	167	269	
	3	220.50	6.33	1395	1694	
	4 (telco)	61.87	6.33	391	391	
	5	26.11	6.33	165	425	
					2955	lb.
2	1	27.79	6.33	176	176	
	2	67.63	6.33	428	530	
	3	153.42	6.33	971	1269	
	4	85.29	6.33	540	540	
					2515	lb.
3	1	27.79	6.33	176	176	
	2	67.63	6.33	428	530	
	3	153.42	6.33	971	1269	
	4	85.29	6.33	540	540	
					2515	lb.
4	1	31.38	6.33	199	458	
	2	49.58	6.33	314	573	
	3	32.58	6.33	206	206	
	4	30.25	6.33	191	294	
	5	145.75	6.33	922	1221	
	6	67.79	6.33	429	688	
	7	74.75	6.33	473	473	
					3913	lb.

EXTERIOR WALLS:

Floor	Wall Number	Final Area (sq. ft.)	Wall Unit Weight (lb./sq. ft.)	Total wt.--wall material only (lb.)	Total wt.--wall+ windows+doors+ louvers (lb.)	
1	1	239.14	16.33	3905	3905	
	2	239.14	16.33	3905	3905	
	3	239.14	16.33	3905	3905	
	4	218.14	16.33	3562	3791	
					15505	lb.
2	1	235.02	16.33	3837	3858	
	2	239.14	16.33	3905	3905	
	3	239.14	16.33	3905	3905	
	4	239.14	16.33	3905	3905	
					15572	lb.
3	1	222.64	16.33	3635	3718	

	2	239.14	16.33	3905	3905
	3	239.14	16.33	3905	3905
	4	239.14	16.33	3905	3905
					15431 lb.
4	1	203.62	16.33	3325	3609
	2	239.14	16.33	3905	3905
	3	239.14	16.33	3905	3905
	4	203.62	16.33	3325	3609
					15027 lb.

FLOOR WEIGHTS:

Floor	Floor Area (sq. ft.)	Unit Weight (lb./sq. ft.)	Additional Weight (lb./sq. ft.)	Total Weight (lb.)
1 (ground)	455.11	62.50	0.00	28444
2	354.74	62.50	0.00	22171
3	354.74	62.50	0.00	22171
4	367.32	63.50	0.00	23325

weight of bent level:

item	Floor Area (sq. ft.)	Unit Weight (lb./sq. ft.)	Total Weight (lb.)
exterior cladding below bent	70.00	4.50	315
exterior cladding above bent	246.40	13.00	3203
catwalk steel decking	233.52	2.72	635
2" catwalk concrete + decking	213.33	34.00	7253
hand rails on catwalk (ft)	331.25	2.27	752
suspended tile ceiling	367.32	1.00	367
suspended gypsum ceiling	77.88	2.50	195
wall on stairs	60.02	6.33	380
ladder to roof			696
catwalk door			165
			13961 lb.

Additional Members	Unit Weight (lb./ft.)	Length (ft.)	Number	Total Weight (lb.)
C 6x8.2	8.20	6.67	2	109
C 6x8.2	8.20	5.83	2	96
C 6x8.2	8.20	5.00	2	82
C 8x11.5	11.50	4.00	2	92
C 8x11.5	11.50	5.50	2	127
C10x15.3	15.30	108.56	1	1661
W 10x49	49.00	4.86	2	476
W 10x15	15.00	49.73	1	746
W 10x68	68.00	4.86	3	992
				4381 lb.

weight of cab floor level:

item	Total Area (sq. ft.)	Unit Weight (lb./sq. ft.)	Total Weight (lb.)
exterior cladding	230.00	7.83	1801
wall on stairs	52.00	6.33	329
flooring	400.00	2.00	800
slab	400.44	40.00	16018
shelf, cabinets, console, sink	---	---	978
			19926 lb.

weight of bottom of glass level:

item	Bottom Length (in.)	Distributed Load (lb./in.)	Total Weight (all 10 panes) (lb.)
3/8" glass	96.00	0.71	681

WPL, MPL for vertical mullions:

Label	Section	Distributed Load (lb./in.)	Weight per Length (k/in)	Mass per Length (k*s2/in2)
COL	CCMOD0	0.71	0.001420	0.00000367

COL	CCMOD1	0.71	0.001420	0.00000367
COL	CCMOD2	0.71	0.001420	0.00000367
COL	S3X7.5A	0.71	0.001420	0.00000367
COL	S3X7.5B	0.71	0.001420	0.00000367
COL	S3X7.5C	0.71	0.001420	0.00000367

weight of top of glass level:

item	Unit Weight (lb./sq. ft.)	Distributed Load (lb./in.)	Total Weight (all 10 panes) (lb.)	
3/8" glass	5.03	0.71	845	lb.

weight of cab roof:

item	total area (sq. ft.)	unit weight (lb./sq. ft.)	Total Weight (lb.)	
exterior cladding	300.00	4.50	1350	
suspended ceiling	609.00	1.00	609	
lights	203.00	4.00	812	
decking	770.00	2.00	1540	
2" rigid insulation	770.00	3.00	2310	
felt and gravel	770.00	10.00	7700	
duckboard (ft), (lb/ft)	80.00	6.56	525	
spiked deck (per George)	---	---	575	
railing	---	---	1000	
		Total Weight: (cladding included)	16421	lb.

STAIR CALCULATIONS:

	Assumptions:		
	1. Light Weight Concrete:	115	lb./cu. ft.
	2. 12 Ga. Steel Decking:	4.27	lb./cu. ft.
	3. 3/8" Cast Alum(Ryerson, pg. 23):	5.29	lb./cu. ft.

Stair Calculations: Floor 1-2

Dwg A3	Stair Tread: Details 1, 2, 3, 4, 5, 6		
	Volume =	0.36	cu. ft.
	Concrete Weight =	42	lb.
	Nosing, Sub-Tread Area=	2.50	sq. ft.
	Nosing Weight =	11	lb.
	Riser Area =	1.80	
	Riser Weight =	7.67	
	Aluminum Tread Area =	0.75	sq. ft.
	Aluminum Weight =	4	lb.
	Angle: 1 1/4x1 1/4x1/8		
	Angle Length =	1.13	ft.
	Angle Weight =	2	lb.
	Tread Weight=	67	lb. (per tread)
	# of treads, floor 2 =	18	treads
Dwg S5	Landing:		
	Concrete Area =	28.75	sq. ft.
	Concrete Weight =	551	lb.
	Metal Deck Weight =	123	lb.
	6x4x1/4 SST		
	Length =	7.67	ft.
	Weight =	120	lb.
	4x4x3/8 SST		
	Length =	12.00	ft.
	Weight =	207	lb.
	Landing Weight =	1001	lb.
	Channels:		
	C10x8.4		
	Length =	47.67	ft.

	Weight =	400	lb.
	Metal Trim (18 Ga)		
	w =	1.02	lb./ft.
	Length =	47.67	ft.
	Weight =	48	lb.
	Channel Weight =	449	lb.
Dwg A3	Railing: Section 3, 6		
	w =	2.27	lb./ft.
	Length =	62.40	ft.
	Weight =	142	lb.
	Posts: Section 78 to 92		
	Length =	17.00	ft.
	Weight =	39	lb.
Stair Calculations: Floor 2-3			
Dwg A3	Stair Tread: Details 1, 2, 3, 4, 5, 6		
	Volume =	0.36	cu. ft.
	Concrete Weight =	42	lb.
	Nosing, Sub-Tread Area=	2.50	sq. ft.
	Nosing Weight =	11	lb.
	Riser Area =	2	
	Riser Weight =	8	
	Aluminum Tread Area =	0.75	sq. ft.
	Aluminum Weight =	4	lb.
	Angle: 1 1/4x1 1/4x1/8		
	Angle Length =	1.13	ft.
	Angle Weight =	2	lb.
	Tread Weight=	67	lb. (per tread)
	# of treads, floor 2 =	18	treads
Dwg S5	Landing:		
	Concrete Area =	28.75	sq. ft.
	Concrete Weight =	551	lb.
	Metal Deck Weight =	123	lb.
	6x4x1/4 SST		
	Length =	7.67	ft.
	Weight =	120	lb.
	4x4x3/8 SST		
	Length =	12.00	ft.
	Weight =	207	lb.
	Landing Weight =	1001	lb.
	Channels:		
	C10x8.4		
	Length =	47.67	ft.
	Weight =	400	lb.
	Metal Trim (18 Ga)		
	w =	1.02	lb./ft.
	Length =	47.67	ft.
	Weight =	48	lb.
	Channel Weight =	449	lb.
Dwg A3	Railing: Section 3, 6		
	w =	2.27	lb./ft.
	Length =	62.40	ft.
	Weight =	142	lb.
	Posts: Section 78 to 92		
	Length =	17.00	ft.

	Weight =	39	lb.
Stair Calculations: Floor 3-4			
Dwg A3	Stair Tread: Details 1, 2, 3, 4, 5, 6		
	Volume =	0.36	cu. ft.
	Concrete Weight =	42	lb.
	Nosing, Sub-Tread Area=	2.50	sq. ft.
	Nosing Weight =	11	lb.
	Riser Area =	2	
	Riser Weight =	8	
	Aluminum Tread Area =	0.75	sq. ft.
	Aluminum Weight =	4	lb.
	Angle: 1 1/4x1 1/4x1/8		
	Angle Length =	1.13	ft.
	Angle Weight =	2	lb.
	Tread Weight=	67	lb. (per tread)
	# of treads, floor 2 =	17	treads
Dwg S5	Landing:		
	Concrete Area =	28.75	sq. ft.
	Concrete Weight =	551	lb.
	Metal Deck Weight =	123	lb.
	6x4x1/4 SST		
	Length =	7.67	ft.
	Weight =	120	lb.
	4x4x3/8 SST		
	Length =	12.00	ft.
	Weight =	207	lb.
	Landing Weight =	1001	lb.
	Channels:		
	C10x8.4		
	Length =	47.67	ft.
	Weight =	400	lb.
	Metal Trim (18 Ga)		
	w =	1.02	lb./ft.
	Length =	47.67	ft.
	Weight =	48	lb.
	Channel Weight =	449	lb.
Dwg A3	Railing: Section 3, 6		
	w =	2.27	lb./ft.
	Length =	62.40	ft.
	Weight =	142	lb.
	Posts: Section 78 to 92		
	Length =	17.00	ft.
	Weight =	39	lb.
Stair Calculations: Floor 4-bent			
Dwg A3	Stair Tread: Details 1, 2, 3, 4, 5, 6		
	Volume =	0.36	cu. ft.
	Concrete Weight =	42	lb.
	Nosing, Sub-Tread Area=	2.50	sq. ft.
	Nosing Weight =	11	lb.
	Riser Area =	2	
	Riser Weight =	8	
	Aluminum Tread Area =	0.75	sq. ft.
	Aluminum Weight =	4	lb.

	Angle: 1 1/4x1 1/4x1/8		
	Angle Length =	1.13	ft.
	Angle Weight =	2	lb.
	Tread Weight=	67	lb. (per tread)
	# of treads, floor 2 =	9	treads
Dwg S5	Landing:		
	Concrete Area =	28.75	sq. ft.
	Concrete Weight =	551	lb.
	Metal Deck Weight =	123	lb.
	6x4x1/4 SST		
	Length =	7.67	ft.
	Weight =	120	lb.
	4x4x3/8 SST		
	Length =	12.00	ft.
	Weight =	207	lb.
	Landing Weight =	1001	lb.
	Channels:		
	C10x8.4		
	Length =	47.67	ft.
	Weight =	400	lb.
	Metal Trim (18 Ga)		
	w =	1.02	lb./ft.
	Length =	47.67	ft.
	Weight =	48	lb.
	Channel Weight =	449	lb.
Dwg A3	Railing: Section 3, 6		
	w =	2.27	lb./ft.
	Length =	62.40	ft.
	Weight =	142	lb.
	Posts: Section 78 to 92		
	Length =	17.00	ft.
	Weight =	39	lb.

APPENDIX B1

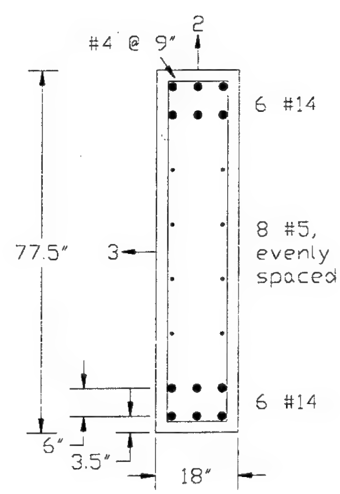
Type L Concrete Bent Column at the Base Evaluation

(based on ACI 318-95)

NOTES : -This sheet is for Rectangular RC Columns
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : Bent at base under thrust and biaxial moment reactions (idealized section at right was analyzed)



the 1 axis is the longitudinal axis out of the page

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

$P_1 := -8.11$ kips $M_1 := 862$ kip·in

$V_2 := 95.1$ kips $M_2 := -3477$ kip·in

$V_3 := -10.7$ kips $M_3 := 38244$ kip·in

$P_1 := -P_1$ tension becomes negative

$P_2 := |V_2|$

$P_3 := |V_3|$ $M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$

ANALYSIS NOTES

- In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.
- The effect of torsion on strength is assumed to be negligible.

SECTION PROPERTIES

$b := 18$	in	
$h := 77.5$	in	
$d2 := 71$	in	depth of reinforcing in 2-2 axis
$d3 := 14.5$	in	depth of reinforcing in 3-3 axis
$A_v := 0.40$	in^2	2-#4 stirrups for shear steel
$s := 9$	in	stirrup spacing
$L := 128.04$	in	length of bent
$K_y := 1.95$		effective length factor

$Ag := b \cdot h$ $\quad in^2$ $\qquad Ag = 1395$ $\quad in^2$

$Iyy := \frac{(h \cdot b^3)}{12}$ $\qquad Iyy = 37665$ $\quad in^4$

MATERIAL PROPERTIES

$f'_c := 4000$ psi

$F_y := 60000$ psi

$E_c := \frac{57000}{1000} \cdot \sqrt{f'_c}$ $\qquad E_c = 3605$ ksi

COMBINED THRUST / MOMENT STRENGTH:

Second order multipliers (ACI 10.13.3):

$$P_{c2} := \frac{\pi^2 \cdot (0.25 \cdot E_c \cdot Iyy)}{(K_y \cdot L)^2}$$

$$\delta_{s2} := \mathrm{if}\left(P_1 > 0, \frac{1.0}{1 - \frac{P_1}{0.75 \cdot P_{c2}}}, 1.0\right) \qquad \delta_{s2} = 1.002$$

$\delta_{s3} := 1$ $\qquad$ slenderness ratio (KL/r)x < 22 (by ACI 318 10.13.2)

$M_{ns3} := 0$ $\qquad M_{s3} := M_3$

$M_{ns2} := 0$ $\qquad M_{s2} := M_2$

$M_3 := M_{ns3} + \delta_{s3} \cdot M_{s3}$ $M_3 = 38244$ kip·in

$M_2 := M_{ns2} + \delta_{s2} \cdot M_{s2}$ $M_2 = 3484$ kip·in

Combined Thrust / Biaxial Moment Interaction - negative moment controls
(from BIAX Analysis, John W. Wallace, Clarkson University, March, 1992)

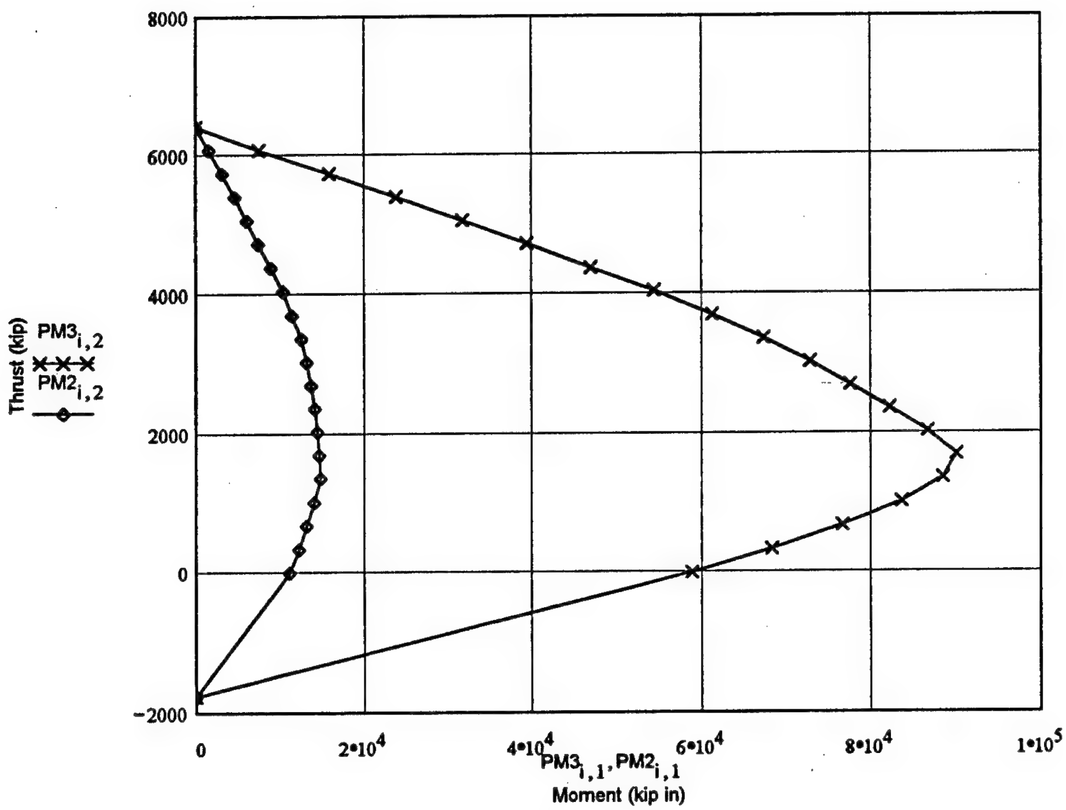
$$PM3 := \begin{bmatrix} 0 & -1769 \\ 58850 & 0 \\ 68290 & 338 \\ 76620 & 674 \\ 83550 & 1012 \\ 88460 & 1350 \\ 90010 & 1688 \\ 86690 & 2025 \\ 82160 & 2363 \\ 77530 & 2699 \\ 72770 & 3037 \\ 67330 & 3374 \\ 61220 & 3711 \\ 54330 & 4050 \\ 46850 & 4386 \\ 39260 & 4725 \\ 31600 & 5062 \\ 23780 & 5400 \\ 15800 & 5737 \\ 7490 & 6075 \\ 0 & 6411 \end{bmatrix} \qquad PM2 := \begin{bmatrix} 0 & -1769 \\ 11090 & 0 \\ 12220 & 338 \\ 13120 & 675 \\ 14000 & 1013 \\ 14780 & 1350 \\ 14610 & 1687 \\ 14370 & 2025 \\ 14100 & 2362 \\ 13650 & 2699 \\ 13110 & 3037 \\ 12490 & 3375 \\ 11410 & 3712 \\ 10310 & 4050 \\ 8886 & 4387 \\ 7458 & 4725 \\ 6042 & 5061 \\ 4628 & 5399 \\ 3174 & 5736 \\ 1554 & 6074 \\ 0 & 6411 \end{bmatrix}$$

$i := 1, 2 .. 21$

ANALYSIS PROCEDURE:

The procedure below finds the ultimate capacity of the section by equating the reactions P1, M2, and M3 to an equivalent force system of the axial load, P1, at eccentricities ecc3=M2/P1 and ecc2=M3/P1. The intersection of the line P=(1/ecc2)M with the curve PM3 defines the nominal axial load Pn2, and likewise with P=(1/ecc3)M, PM2, and Pn3. The values of Pn2, Pn3, and Po (ultimate strength in tension or compression at zero moment) are then combined using the reciprocal load method.

To find Pn2, the "if" statements below step through the PM3 curve starting from the point of ultimate tensile capacity with zero moment (negative P) until the point on the curve (M1,P1), the first point above where the line P=(1/ecc2)M. The point below the intersection of P=(1/ecc2)M and the PM3 curve is found as (M2,P2). The equation of the line between (M1,P1) and (M2,P2) is defined by "slope" and "intcp", the slope and P-axis intercept of this line, receptively. A linear interpolation between the two lines calculates the value of Pn2 where they intersect. Pn3 is similarly computed.



Inverse Eccentricities: (e=1/ecc)

$e_2 := \dfrac{P_1}{M_3}$ $\quad e_2 = 0.00021 \quad in^{-1}$

$e_3 := \dfrac{P_1}{M_2}$ $\quad e_3 = 0.00233 \quad in^{-1}$

Finding P_{n2} due to M_3:

$P1_3 := \mathrm{if}\left(PM3_{1,2} > e_2 \cdot PM3_{1,1}, PM3_{1,2}, 1\right)$ $\quad P1_3 = 1$

$P1_3 := \mathrm{if}\left(PM3_{2,2} > e_2 \cdot PM3_{2,1}, PM3_{2,2}, 1\right)$ $\quad P1_3 = 1$

$P1_3 := \mathrm{if}\left(PM3_{3,2} > e_2 \cdot PM3_{3,1}, PM3_{3,2}, 1\right)$ $\quad P1_3 = 338$ kips

$P2_3 := PM3_{2,2}$ $\quad P2_3 = 0$ kips

$M1_3 := PM3_{3,1}$ $\quad M1_3 = 68290$ kip·in

$M2_3 := PM3_{2,1}$ $\quad M2_3 = 58850$ kip·in

Linearly interpolate for P_{n2}

$$slope := if\left(M2_3 > M1_3, \frac{P2_3 - P1_3}{M2_3 - M1_3}, \frac{P1_3 - P2_3}{M1_3 - M2_3}\right) \qquad slope = 0.0358$$

$$intcp := P1_3 - slope \cdot M1_3 \qquad intcp = -2107$$

$$P_{n2} := \frac{\frac{-intcp}{slope}}{\frac{1}{e_2} - \frac{1}{slope}} \qquad P_{n2} = 12.6 \quad \text{kips}$$

Finding P_{n3} due to M_2:

$$P1_2 := if\left(PM2_{1,2} > e_3 \cdot PM2_{1,1}, PM2_{1,2}, 1\right) \qquad P1_2 = 1$$

$$P1_2 := if\left(PM2_{2,2} > e_3 \cdot PM2_{2,1}, PM2_{2,2}, 1\right) \qquad P1_2 = 1$$

$$P1_2 := if\left(PM2_{3,2} > e_3 \cdot PM2_{3,1}, PM2_{3,2}, 1\right) \qquad P1_2 = 338 \quad \text{kips}$$

$$P2_2 := PM2_{2,2} \qquad P2_2 = 0 \quad \text{kips}$$

$$M1_2 := PM2_{3,1} \qquad M1_2 = 12220 \quad \text{kip} \cdot \text{in}$$

$$M2_2 := PM2_{2,1} \qquad M2_2 = 11090 \quad \text{kip} \cdot \text{in}$$

linearly interpolate for P_{n3}

$$slope := if\left(M2_2 > M1_2, \frac{P2_2 - P1_2}{M2_2 - M1_2}, \frac{P1_2 - P2_2}{M1_2 - M2_2}\right) \qquad slope = 0.299$$

$$intcp := P1_2 - slope \cdot M1_2 \qquad intcp = -3317$$

$$P_{n3} := \frac{\frac{-intcp}{slope}}{\frac{1}{e_3} - \frac{1}{slope}} \qquad P_{n3} = 26 \quad \text{kips}$$

$$P_{no} := if\left(P_1 > 0, PM3_{21,2}, PM3_{1,2}\right) \qquad P_{no} = 6411 \quad \text{kips}$$

Reciprocal load method (ref. ACI R10.3.6)

$$P_n := \frac{1}{\frac{1}{P_{n3}} + \frac{1}{P_{n2}} - \frac{1}{P_{no}}} \qquad P_n = 8.48 \quad \text{kips}$$

SHEAR:

Individual capacity values:

$$V_{c3} := 2\cdot\left(1 + \frac{1}{2000}\cdot\frac{P_1}{Ag}\right)\cdot\frac{\sqrt{f'_c}\cdot h\cdot d3}{1000}$$ ACI 318 - EQ(11-4)

$$V_{c2} := 2\cdot\left(1 + \frac{1}{2000}\cdot\frac{P_1}{Ag}\right)\cdot\frac{\sqrt{f'_c}\cdot b\cdot d2}{1000}$$ ACI 318 - EQ(11-4)

$$V_{s3} := \frac{A_v\cdot F_y\cdot d3}{s\cdot 1000}$$ ACI 318 - EQ(11-15)

$$V_{s2} := \frac{A_v\cdot F_y\cdot d2}{s\cdot 1000}$$ ACI 318 - EQ(11-15)

Combined Shear Capacities:

$$V_{n2} := V_{c2} + V_{s2}$$ $V_{n2} = 351$ kips

$$V_{n3} := V_{c3} + V_{s3}$$ $V_{n3} = 181$ kips

DEMAND CAPACITY RATIO (DCR)

The demand capacity ratio is the maximum of the thrust and shear as defined below.

$$DCR := \left[\frac{\frac{P_1}{P_n}}{\sqrt{\left(\frac{P_2}{V_{n2}}\right)^2 + \left(\frac{P_3}{V_{n3}}\right)^2}}\right]$$

$\max(DCR) = 0.956$

APPENDIX B2

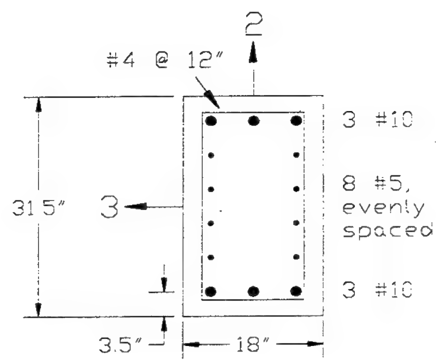
Type L Concrete Bent Column at the Top of Shaft Evaluation

(based on ACI 318-95)

NOTES : -This sheet is for Rectangular RC Columns
-Note member axis definitions below.

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : Bent at shaft under thrust and biaxial moment reactions (idealized section at right was analyzed)



the 1 axis is the longitudinal axis out of the page

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

$P_1 := 51.1$ kips $M_1 := -440$ kip·in

$V_2 := 83.0$ kips $M_2 := -377$ kip·in

$V_3 := 3.2$ kips $M_3 := -8910$ kip·in

$P_1 := -P_1$ tension becomes negative

$P_2 := V_2$

$P_3 := |V_3|$ $M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$

ANALYSIS NOTES

- In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.
- The effect of torsion on strength is assumed to be negligible.

SECTION PROPERTIES

$b := 18$ in

$h := 31.5$ in

$d2 := 28$ in — depth of reinforcing in 2-2 axis

$d3 := 14.5$ in — depth of reinforcing in 3-3 axis

$A_v := 0.40$ in^2 — 2-#4 stirrups for shear steel

$s := 12$ in — stirrup spacing

$L := 156$ in — length of bent

$K_x := 2.30$ — effective length factor

$K_y := 1.95$ — effective length factor

$Ag := b \cdot h$ in^2 — $Ag = 567$ in^2

$Ixx := \frac{(b \cdot h^3)}{12}$ — $Ixx = 46884$ in^4

$Iyy := \frac{(h \cdot b^3)}{12}$ — $Iyy = 15309$ in^4

MATERIAL PROPERTIES

$f'_c := 4000$ psi

$F_y := 60000$ psi

$E_c := \frac{57000}{1000} \cdot \sqrt{f'_c}$ — $E_c = 3605$ ksi

COMBINED THRUST / MOMENT STRENGTH:

Second order moment multipliers:

$$P_{c3} := \frac{\pi^2 \cdot (0.25 \cdot E_c \cdot Iyy)}{(K_y \cdot L)^2}$$

$$P_{c2} := \frac{\pi^2 \cdot (0.25 \cdot E_c \cdot Ixx)}{(K_x \cdot L)^2}$$

$$\delta_{s3} := \text{if}\left(P_1 > 0, \frac{1.0}{1 - \frac{P_1}{0.75 \cdot P_{c3}}}, 1.0\right) \qquad \delta_{s3} = 1$$

$$\delta_{s2} := \text{if}\left(P_1 > 0, \frac{1.0}{1 - \frac{P_1}{0.75 \cdot P_{c2}}}, 1.0\right) \qquad \delta_{s2} = 1$$

$M_{ns3} := 0 \qquad M_{s3} := M_3$

$M_{ns2} := 0 \qquad M_{s2} := M_2$

$M_3 := M_{ns3} + \delta_{s3} \cdot M_3 \qquad M_3 = 8910$ kip·in

$M_2 := M_{ns2} + \delta_{s2} \cdot M_2 \qquad M_2 = 377$ kip·in

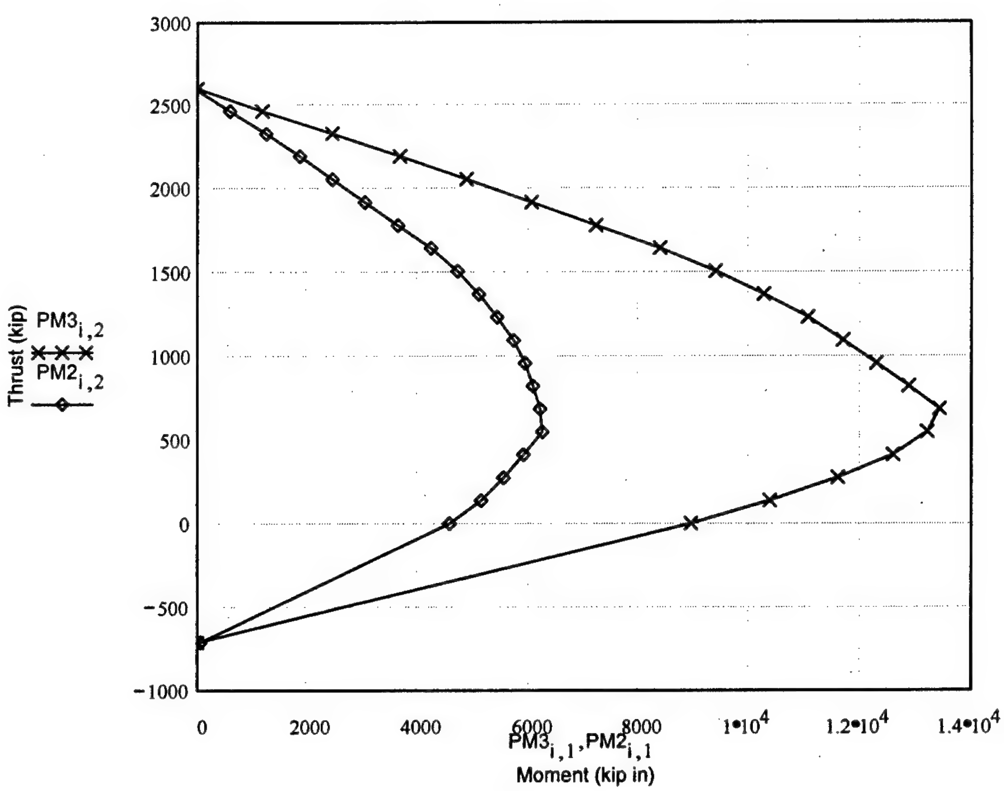
Combined Thrust / Biaxial Moment Interaction - negative moment controls
(from BIAX Analysis, John W. Wallace, Clarkson University, March, 1992)

$$PM3 := \begin{bmatrix} 0 & -710 \\ 8939 & 0 \\ 10370 & 137 \\ 11600 & 274 \\ 12590 & 410 \\ 13200 & 547 \\ 13420 & 684 \\ 12870 & 820 \\ 12290 & 957 \\ 11690 & 1094 \\ 11060 & 1231 \\ 10260 & 1367 \\ 9389 & 1504 \\ 8380 & 1641 \\ 7220 & 1778 \\ 6055 & 1914 \\ 4881 & 2051 \\ 3681 & 2188 \\ 2453 & 2325 \\ 1179 & 2461 \\ 0 & 2598 \end{bmatrix} \qquad i := 1, 2 .. 21 \qquad PM2 := \begin{bmatrix} 56 & -710 \\ 4579 & 0 \\ 5143 & 137 \\ 5545 & 273 \\ 5911 & 410 \\ 6247 & 547 \\ 6209 & 684 \\ 6083 & 820 \\ 5935 & 957 \\ 5732 & 1094 \\ 5433 & 1230 \\ 5107 & 1367 \\ 4720 & 1504 \\ 4243 & 1641 \\ 3646 & 1778 \\ 3050 & 1914 \\ 2457 & 2051 \\ 1865 & 2188 \\ 1254 & 2324 \\ 583 & 2461 \\ -53 & 2598 \end{bmatrix}$$

ANALYSIS PROCEDURE:

The procedure below finds the ultimate capacity of the section by equating the reactions P1, M2, and M3 to an equivalent force system of the axial load, P1, at eccentricities ecc3=M2/P1 and ecc2=M3/P1. The intersection of the line P=(1/ecc2)M with the curve PM3 defines the nominal axial load Pn2, and likewise with P=(1/ecc3)M, PM2, and Pn3. The values of Pn2, Pn3, and Po (ultimate strength in tension or compression at zero moment) are then combined using the reciprocal load method.

To find Pn2, the "if" statements below step through the PM3 curve starting from the point of ultimate tensile capacity with zero moment (negative P) until the point on the curve (M1,P1), the first point above where the line P=(1/ecc2)M. The point below the intersection of P=(1/ecc2)M and the PM3 curve is found as (M2,P2). The equation of the line between (M1,P1) and (M2,P2) is defined by "slope" and "intcp", the slope and P-axis intercept of this line, receptively. A linear interpolation between the two lines calculates the value of Pn2 where they intersect. Pn3 is similarly computed.



Inverse Eccentricities: (e=1/ecc)

$e_2 := \dfrac{P_1}{M_3}$ $\quad e_2 = -0.00574 \ \ in^{-1}$

$e_3 := \dfrac{P_1}{M_2}$ $\quad e_3 = -0.13554 \ \ in^{-1}$

Finding P_{n2} due to M_3:

$P1_3 := \mathrm{if}\left(PM3_{1,2} > e_2 \cdot PM3_{1,1}, PM3_{1,2}, 1\right)$ $P1_3 = 1$

$P1_3 := \mathrm{if}\left(PM3_{2,2} > e_2 \cdot PM3_{2,1}, PM3_{2,2}, 1\right)$ $P1_3 = 0$ kips

$P2_3 := PM3_{1,2}$ $P2_3 = -710$ kips

$M1_3 := PM3_{2,1}$ $M1_3 = 8939$ kip·in

$M2_3 := PM3_{1,1}$ $M2_3 = 0$ kip·in

Linearly interpolate for P_{n2}

$$slope := \mathrm{if}\left(M2_3 > M1_3, \frac{P2_3 - P1_3}{M2_3 - M1_3}, \frac{P1_3 - P2_3}{M1_3 - M2_3}\right) \qquad slope = 0.079$$

$$intcp := P1_3 - slope \cdot M1_3 \qquad intcp = -710$$

$$P_{n2} := \frac{\dfrac{-intcp}{slope}}{\dfrac{1}{e_2} - \dfrac{1}{slope}} \qquad P_{n2} = -47.8 \text{ kips}$$

Finding P_{n3} due to M_2:

$P1_2 := \mathrm{if}\left(PM2_{1,2} > e_3 \cdot PM2_{1,1}, PM2_{1,2}, 1\right)$ $P1_2 = 1$

$P1_2 := \mathrm{if}\left(PM2_{2,2} > e_3 \cdot PM2_{2,1}, PM2_{2,2}, 1\right)$ $P1_2 = 0$ kips

$P2_2 := PM2_{1,2}$ $P2_2 = -710$ kips

$M1_2 := PM2_{2,1}$ $M1_2 = 4579$ kip·in

$M2_2 := PM2_{1,1}$ $M2_2 = 56$ kip·in

linearly interpolate for P_{n2}

$$slope := \mathrm{if}\left(M2_2 > M1_2, \frac{P2_2 - P1_2}{M2_2 - M1_2}, \frac{P1_2 - P2_2}{M1_2 - M2_2}\right) \qquad slope = 0.157$$

$$intcp := P1_2 - slope \cdot M1_2 \qquad intcp = -710$$

$$P_{n3} := \frac{\dfrac{-intcp}{slope}}{\dfrac{1}{e_3} - \dfrac{1}{slope}} \qquad P_{n3} = -333.1 \text{ kips}$$

$$P_{no} := \text{if}\left(P_1 > 0, PM3_{21,2}, PM3_{1,2}\right) \qquad P_{no} = -710 \quad \text{kips}$$

Reciprocal load method (ref. ACI R10.3.6)

$$P_n := \frac{1}{\dfrac{1}{P_{n3}} + \dfrac{1}{P_{n2}} - \dfrac{1}{P_{no}}} \qquad P_n = -44.4 \quad \text{kips}$$

SHEAR:

Individual capacity values:

$$V_{c3} := 2\cdot\left(1 + \frac{1}{2000}\cdot\frac{P_1}{Ag}\right)\cdot\frac{\sqrt{f'_c}\cdot h\cdot d3}{1000} \qquad \text{ACI 318 - EQ(11-4)}$$

$$V_{c2} := 2\cdot\left(1 + \frac{1}{2000}\cdot\frac{P_1}{Ag}\right)\cdot\frac{\sqrt{f'_c}\cdot b\cdot d2}{1000} \qquad \text{ACI 318 - EQ(11-4)}$$

$$V_{s3} := \frac{A_v\cdot F_y\cdot d3}{s\cdot 1000} \qquad \text{ACI 318 - EQ(11-15)}$$

$$V_{s2} := \frac{A_v\cdot F_y\cdot d2}{s\cdot 1000} \qquad \text{ACI 318 - EQ(11-15)}$$

Combined Shear Capacities:

$$V_{n2} := V_{c2} + V_{s2} \qquad V_{n2} = 119.7 \quad \text{kips}$$

$$V_{n3} := V_{c3} + V_{s3} \qquad V_{n3} = 86.8 \quad \text{kips}$$

DEMAND CAPACITY RATIO (DCR)

The demand capacity ratio is the maximum of the thrust and shear as defined below.

$$DCR := \left[\frac{P_1}{P_n} \quad \sqrt{\left(\frac{P_2}{V_{n2}}\right)^2 + \left(\frac{P_3}{V_{n3}}\right)^2}\right] \qquad \max(DCR) = 1.15$$

APPENDIX B3

Type L Concrete Bent Beam at the Top of Shaft Evaluation

(based on ACI 318-95)

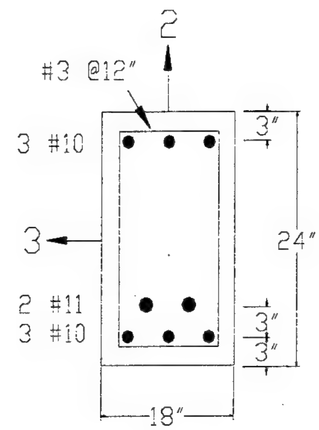
NOTES : -This sheet is for Rectangular RC Members
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : Bent at top of shaft (dwg, S1)

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight



the 1 axis is the longitudinal axis out of the page

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 52.5$ kips $\qquad M_1 := 382$ kip·in

$V_2 := -55.2$ kips $\qquad M_2 := 430$ kip·in

$V_3 := 2.5$ kips $\qquad M_3 := -8290$ kip·in

$P_1 := -P_1$ $\qquad$ tension becomes negative

$P_2 := |V_2|$

$P_3 := |V_3|$ $\qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

- The effect of torsion on strength is assumed to be negligible.

SECTION PROPERTIES

$b := 18$ in

$h := 24$ in

$d := 21$ in depth of steel

$s := 12$ in stirrup spacing

$Ag := b \cdot h$ in^2 $Ag = 432$ in^2

MATERIAL PROPERTIES

$f'_c := 4000$ psi

$f_y := 60000$ psi

CALCULATED PROPERTIES

$A_v := 2 \cdot 0.11$ in^2 (2-#3 stirrups for shear steel)

$A_s := 6 \cdot 1.27 + 2 \cdot 1.56$ in^2 (2-#11 and 6-#11 steel bars for reinforcing)

COMBINED THRUST / MOMENT STRENGTH:

ANALYSIS PROCEDURE:

The procedure below finds the ultimate capacity of the section by equating the reactions P1, M2, and M3 to an equivalent force system of the axial load, P1, at eccentricities ecc3=M2/P1 and ecc2=M3/P1. The intersection of the line P=(1/ecc2)M with the curve PM3 defines the nominal axial load Pn2, and likewise with P=(1/ecc3)M, PM2, and Pn3. The values of Pn2, Pn3, and Po (ultimate strength in tension or compression at zero moment) are then combined using the reciprocal load method.

To find Pn2, the "if" statements below step through the PM3 curve starting from the point of ultimate tensile capacity with zero moment (negative P) until the point on the curve (M1,P1), the first point above where the line P=(1/ecc2)M. The point below the intersection of P=(1/ecc2)M and the PM3 curve is found as (M2,P2). The equation of the line between (M1,P1) and (M2,P2) is defined by "slope" and "intcp", the slope and P-axis intercept of this line, receptively. A linear interpolation between the two lines calculates the value of Pn2 where they intersect. Pn3 is similarly computed.

Combined Thrust / Biaxial Moment Interaction - negative moment controls
(from BIAX Analysis, John W. Wallace, Clarkson University, March, 1992)

$$PM3 := \begin{bmatrix} -1123 & -644 \\ 4575 & 0 \\ 5388 & 109 \\ 6185 & 219 \\ 6944 & 328 \\ 7658 & 437 \\ 8228 & 547 \\ 8447 & 656 \\ 8177 & 765 \\ 7896 & 875 \\ 7597 & 984 \\ 7237 & 1093 \\ 6784 & 1202 \\ 6252 & 1312 \\ 5527 & 1421 \\ 4795 & 1530 \\ 4057 & 1639 \\ 3311 & 1749 \\ 2553 & 1858 \\ 1776 & 1967 \\ 1060 & 2077 \\ 0 & 2077 \end{bmatrix}$$

$$j := 1, 2 .. 22$$

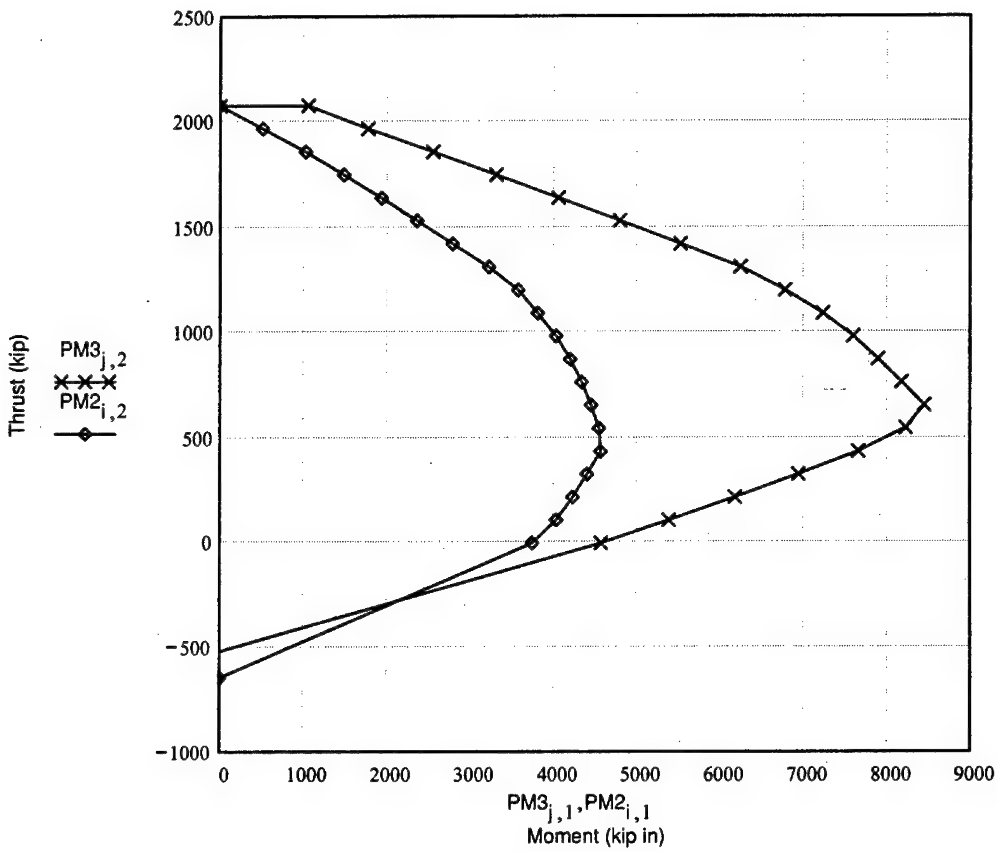
$$i := 1, 2 .. 21$$

$$PM2 := \begin{bmatrix} 0 & -644 \\ 3745 & 0 \\ 4033 & 109 \\ 4230 & 219 \\ 4404 & 328 \\ 4566 & 437 \\ 4549 & 546 \\ 4454 & 656 \\ 4338 & 765 \\ 4200 & 874 \\ 4028 & 984 \\ 3807 & 1093 \\ 3572 & 1202 \\ 3222 & 1312 \\ 2789 & 1421 \\ 2361 & 1530 \\ 1935 & 1639 \\ 1487 & 1749 \\ 1030 & 1858 \\ 515 & 1967 \\ 0 & 2077 \end{bmatrix}$$

Inverse Eccentricities: (e=1/ecc)

$$e_2 := \frac{P_1}{M_3} \qquad e_2 = -0.00633 \quad in^{-1}$$

$$e_3 := \frac{P_1}{M_2} \qquad e_3 = -0.12209 \quad in^{-1}$$



Finding P_{n2} due to M_3:

$P1_3 := \text{if}\left(PM3_{1,2} > e_2 \cdot PM3_{1,1}, PM3_{1,2}, 1\right)$ $P1_3 = 1$

$P1_3 := \text{if}\left(PM3_{2,2} > e_2 \cdot PM3_{2,1}, PM3_{2,2}, 1\right)$ $P1_3 = 0$ kips

$P2_3 := PM3_{1,2}$ $P2_3 = -644$ kips

$M1_3 := PM3_{2,1}$ $M1_3 = 4.575 \cdot 10^3$ kip·in

$M2_3 := PM3_{1,1}$ $M2_3 = -1.123 \cdot 10^3$ kip·in

linearly interpolate for P_{n2}

$$slope := \text{if}\left(M2_3 > M1_3, \frac{P2_3 - P1_3}{M2_3 - M1_3}, \frac{P1_3 - P2_3}{M1_3 - M2_3}\right) \quad slope = 0.113$$

$intcp := P1_3 - slope \cdot M1_3$ $intcp = -517.076$

$$P_{n2} := \frac{\frac{-\text{intcp}}{\text{slope}}}{\frac{1}{e_2} - \frac{1}{\text{slope}}} \qquad P_{n2} = -27.436 \quad \text{kips}$$

Finding P_{n3} due to M_2:

$$P1_2 := \text{if}\left(PM2_{1,2} > e_3 \cdot PM2_{1,1}, PM2_{1,2}, 1\right) \qquad P1_2 = 1$$

$$P1_2 := \text{if}\left(PM2_{2,2} > e_3 \cdot PM2_{2,1}, PM2_{2,2}, 1\right) \qquad P1_2 = 0 \quad \text{kips}$$

$$P2_2 := PM2_{1,2} \qquad P2_2 = -644 \quad \text{kips}$$

$$M1_2 := PM2_{2,1} \qquad M1_2 = 3745 \quad \text{kip} \cdot \text{in}$$

$$M2_2 := PM2_{1,1} \qquad M2_2 = 0 \quad \text{kip} \cdot \text{in}$$

linearly interpolate for P_{n3}

$$\text{slope} := \text{if}\left(M2_2 > M1_2, \frac{P2_2 - P1_2}{M2_2 - M1_2}, \frac{P1_2 - P2_2}{M1_2 - M2_2}\right) \qquad \text{slope} = 0.172$$

$$\text{intcp} := P1_2 - \text{slope} \cdot M1_2$$

$$P_{n3} := \frac{\frac{-\text{intcp}}{\text{slope}}}{\frac{1}{e_3} - \frac{1}{\text{slope}}} \qquad P_{n3} = -267.391 \quad \text{kips}$$

$$P_{no} := \text{if}\left(P_1 > 0, PM3_{21,2}, PM3_{1,2}\right) \qquad P_{no} = -644 \quad \text{kips}$$

Reciprocal load method (ref. ACI R10.3.6)

$$P_n := \frac{1}{\frac{1}{P_{n3}} + \frac{1}{P_{n2}} - \frac{1}{P_{no}}} \qquad P_n = -25.883 \quad \text{kips}$$

SHEAR

Individual capacity values:

$$V_{c3} := 2 \cdot \left(1 + \frac{P_1}{500 \cdot Ag}\right) \cdot \frac{\sqrt{f'_c} \cdot h \cdot b}{1000} \qquad \text{(ACI 318, EQ 11-8)}$$

$$V_{c2} := 2 \cdot \left(1 + \frac{P_1}{500 \cdot Ag}\right) \cdot \frac{\sqrt{f'_c} \cdot b \cdot h}{1000} \qquad \text{(ACI 318, EQ 11-8)}$$

$$V_{s3} := \frac{A_v \cdot f_y \cdot b}{s \cdot 1000}$$ ACI 318 - EQ(11-15)

$$V_{s2} := \frac{A_v \cdot f_y \cdot d}{s \cdot 1000}$$ ACI 318 - EQ(11-15)

Combined Shear Capacities:

$V_{n2} := V_{c2} + V_{s2}$ $\quad V_{n2} = 77.731$ kips

$V_{n3} := V_{c3} + V_{s3}$ $\quad V_{n3} = 74.431$ kips

DEMAND CAPACITY RATIO (DCR)

The demand capacity ratio is the maximum of the thrust and shear as defined below.

$$DCR := \left[\frac{P_1}{P_n} \quad \sqrt{\left(\frac{P_2}{V_{n2}}\right)^2 + \left(\frac{P_3}{V_{n3}}\right)^2} \right]$$ $\max(DCR) = 2.028$

APPENDIX B4

Type L Concrete Bent Connection at the Top of Shaft Evaluation

(based on AISC-LRFD. 2nd ed., 1994 and ACI 318-95)

NOTES : -This sheet is for the pin connection at the intersection of the concrete bents at the top of the shaft
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

CONNECTION : Pin connection at top of shaft (dwg. S-1)

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

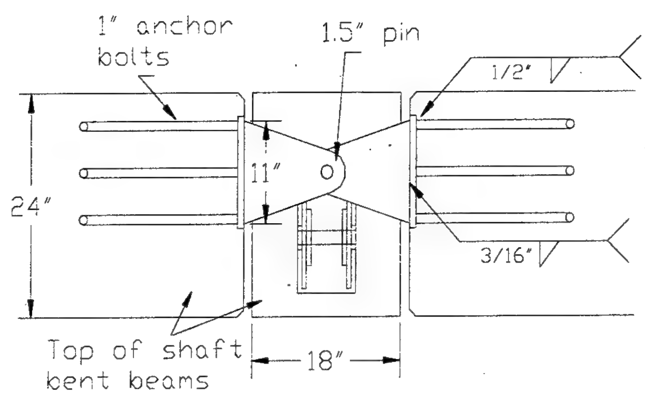
P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -20.2\,\text{kips}$ $M_1 := 383$ kip·in

$V_2 := 45.4$ kips $M_2 := 0$ kip·in

$V_3 := 3.3$ kips $M_3 := 0$ kip·in



$P_1 := |P_1|$

$P_2 := |V_2|$

$P_3 := |V_3|$ $M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$.

ANALYSIS NOTES

-This sheet consists of the calculations for finding the controlling strength of the pin connection.

SECTION PROPERTIES

$d_b := 1.0$ in

$d_{pin} := 1.5$ in

MATERIAL & CODE PROPERTIES

$f'_c := 4000$	psi	$\alpha := 1.0$
$F_{y\_rebar} := 60000$	psi	$\beta := 1$
$F_w := 70000$	psi	$\lambda := 1$
$F_y := 36000$	psi	$\phi_v := 0.75$
$F_u := 58000$	psi	$\phi_y := 0.90$

CALCULATED PROPERTIES

$A_{bar} := \frac{\pi}{4} \cdot d_b^2$

$A_{pin} := \frac{\pi}{4} \cdot d_{pin}^2$

$A_{g\_min} := 5.5 \cdot 0.5$ $\quad A_{g\_min} = 2.75$ in^2 (minimum plate area)

AXIAL : Tension in Anchor Bolts

Development length for a bar: (ACI 12.2.2)

$$l_d := d_b \cdot \left(\frac{F_{y\_rebar} \cdot \alpha \cdot \beta \cdot \lambda}{20 \sqrt{f'_c}} \right) \qquad l_d = 47.4 \quad \text{in}$$

Allowable Force:

$$P_{tot} := 6 \cdot A_{bar} \cdot F_{y\_rebar} \qquad \frac{P_{tot}}{1000} = 283 \quad \text{kips}$$

$$P_a := \frac{12}{l_d} \cdot \frac{P_{tot}}{1000} \qquad P_a = 71.5 \quad \text{kips}$$

SHEAR :

Shear on welds:

On the bolts:

$$R_{n\_bw} := (0.707 \cdot .5 \cdot \pi \cdot d_b \cdot 0.6 \cdot F_w) \cdot 6 \qquad \frac{\phi_v \cdot R_{n\_bw}}{1000} = 210 \quad \text{kips}$$

On the plates:

- 3/16" welds assumed
- 11 inch long welds, both sides of the plate, 2 plates

$$R_{n\_plw} := 0.707 \cdot \frac{3}{16} \cdot F_w \cdot 11 \cdot 2 \cdot 2 \qquad \phi_v \cdot \frac{R_{n\_plw}}{1000} = 306 \quad \text{kips}$$

Shear on pin:

-Threads are excluded from the shear plane.
-Bolts are in double shear.

$$R_{n\_pin} := F_y \cdot A_{pin} \cdot 2 \qquad \phi_v \cdot \frac{R_{n\_pin}}{1000} = 95.4 \quad \text{kips}$$

Shear on plates:

- 2 plates total

$$R_{n\_pl} := 0.60 \cdot A_{g\_min} \cdot F_y \cdot 2 \qquad \phi_y \cdot \frac{R_{n\_pl}}{1000} = 107 \quad \text{kips}$$

BEARING:

- 2 plates total

$$R_{n\_bear} := 2.4 \cdot d_{pin} \cdot 0.5 \cdot F_u \cdot 2 \qquad \phi_v \cdot \frac{R_{n\_bear}}{1000} = 157 \quad \text{kips}$$

For shear, pin controlls: $R_n := \frac{\phi_v \cdot R_{n\_pin}}{1000}$

DEMAND CAPACITY RATIO (DCR):

$$DCR := \begin{bmatrix} \frac{P_1}{P_a} \\ \frac{P_2}{R_n} \end{bmatrix} \qquad \max(DCR) = 0.476$$

APPENDIX B5

Type L Concrete Bent-to-Roof Beam Connection Evaluation

(based on AISC-LRFD. 2nd ed., 1994)

NOTES : -This sheet checks the strength of the roof beam to bent connection at the top of the shaft

---

TOWER : Type L, 50 ft. (Salinas, CA)

CONNECTION : Typ.shear connection - Bent to Roof Beam (dwg. S-5)

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

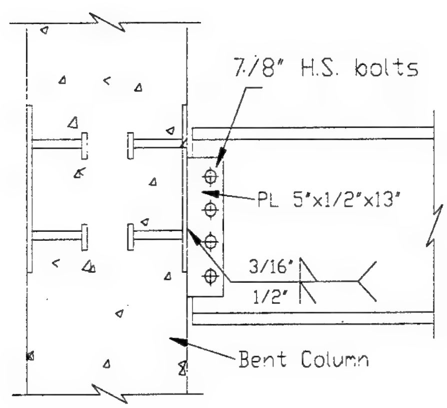
P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -27.8\,\text{kips}$ $\quad M_1 := 8 \;\; \text{kip·in}$

$V_2 := -50.8\,\text{kips}$ $\quad M_2 := 0 \;\; \text{kip·in}$

$V_3 := -5.2\,\text{kips}$ $\quad M_3 := 0 \;\; \text{kip·in}$



$P_2 := |V_2|$

$P_3 := |V_3|$ $\qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

SECTION PROPERTIES

$d_b := 0.875$ in

$t_p := 0.5$ in

$l_p := 13$ in

MATERIAL & CODE PROPERTIES

$F_{u\_b} := 48$ ksi $\qquad \phi_v := 0.75$

$F_w := 70$ ksi $\qquad \phi_y := 0.90$

$F_y := 36$ ksi $\qquad \phi_{bw} := 0.80$

$\phi_{fw} := 0.75$

CALCULATED PROPERTIES

$$A_b := \frac{\pi}{4} \cdot d_b^{\,2}$$

$$A_g := l_p \cdot t_p$$

SHEAR :

Shear on welds (1/2" butt weld + 3/16" fillet weld):

$$\phi R_{n\_w} := \phi_{bw} \cdot A_g \cdot 0.6 \cdot F_w + \phi_{fw} \cdot 0.707 \cdot \frac{3}{16} \cdot l_p \cdot 0.6 \cdot F_w$$

$\phi R_{n\_w} = 273$ kips

Shear on bolts:

- 4 bolts total

$R_{n\_b} := A_b \cdot F_{u\_b} \cdot 4 \qquad \phi_v \cdot R_{n\_b} = 86.6$ kips

Shear on plate:

$R_{n\_pl} := 0.60 \cdot A_g \cdot F_y \qquad \phi_y \cdot R_{n\_pl} = 126$ kips

STRENGTH:

Shear of bolts controls:

$$R_n := \phi_v \cdot R_{n\_b}$$

$DCR := \dfrac{P_2}{R_n} \qquad DCR = 0.587$

APPENDIX B6

Type L Cab Corner Mullion (Columns 1 - 4) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

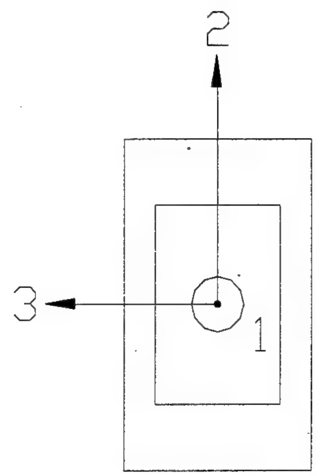
NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : mullion @ base

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight



the 1 axis is the longitudinal axis out of the page

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -36.9\,\text{kips}$ $\qquad M_1 := 55\ \text{kip}\cdot\text{in}$

$V_2 := -20.6\,\text{kips}$ $\qquad M_2 := -246\ \text{kip}\cdot\text{in}$

$V_3 := 12.8\ \text{kips}$ $\qquad M_3 := -714\ \text{kip}\cdot\text{in}$

$P_2 := |V_2|$

$P_3 := |V_3|$ $\qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$d := 7$	in	$k_{nt3} := 0.8$	K values by LRFD Table C-C2.1
$t_w := 0.5$	in	$k_{nt2} := 0.8$	
$b_f := 4.0$	in	$k_{lt3} := 2.10$	
$t_f := 1.0$	in	$k_{lt2} := 2.10$	
$J := 50$	in^4	$L_{b3} := 37.9\ in$	
		$L_{b2} := 37.9\ in$	

MATERIAL & CODE PROPERTIES

$E := 29000\ ksi$	$Fy := 36\ ksi$	$\phi_b := 0.90$	$\phi_c := 0.85$
$G := 11200\ ksi$	$Fu := 58\ ksi$	$\phi_v := 0.90$	$C_b := 1.0$
$Fr := 10\ ksi$	$Fyf := Fy$	$\phi_{ty} := 0.90$	
$F_L := Fyf - Fr$	$Fyw := Fy$	$\phi_{tf} := 0.75$	

CALCULATED PROPERTIES

$$A := \left[b_f \cdot t_f + \left(d - 2 \cdot t_f\right) \cdot t_w\right] \cdot 2$$

$A = 13 \quad in^2$

$A_e := A$

$$I_3 := b_f \cdot \frac{d^3}{12} - \frac{\left(b_f - 2 \cdot t_w\right) \cdot \left(d - 2 \cdot t_f\right)^3}{12} \qquad I_3 = 83.1 \quad in^4$$

$$I_2 := \frac{d \cdot b_f^{\,3}}{12} - \frac{\left(b_f - 2 \cdot t_w\right)^3 \cdot \left(d - 2 \cdot t_f\right)}{12} \qquad I_2 = 26.1 \quad in^4$$

$$S_3 := \frac{I_3}{\left(\frac{d}{2}\right)} \qquad S_3 = 23.7 \quad in^3$$

$$S_2 := \frac{I_2}{\left(\frac{b_f}{2}\right)} \qquad S_2 = 13 \quad in^3$$

$$Z_3 := \left[(b_f \cdot t_f)\cdot\left(\frac{d}{2} - \frac{t_f}{2}\right) + 2\cdot\left[t_w\cdot\left(\frac{d}{2} - t_f\right)\right]\cdot\frac{\left(\frac{d}{2} - t_f\right)}{2}\right]\cdot 2 \qquad Z_3 = 30.3 \quad in^3$$

$$Z_2 := \left[(d - 2\cdot t_f)\cdot t_w\cdot\left(\frac{b_f}{2} - \frac{t_w}{2}\right) + \frac{b_f}{2}\cdot t_f\cdot\frac{b_f}{4}\cdot 2\right]\cdot 2 \qquad Z_2 = 16.8 \quad in^3$$

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := 2\cdot d\cdot t_w$$

$$A_f := 2\cdot b_f\cdot t_f$$

$$h := d - 3\cdot t_f$$

$$r_2 := \sqrt{\frac{I_2}{A}} \qquad b := b_f - 3\cdot t_w$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, (\phi_{tf}\cdot Fu\cdot A_e), \phi_{ty}\cdot Fy\cdot A\right]$$ (eq. LRFD D1-1) (eq. LRFD D1-2)

$\phi P_{nt} = 421$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{p\_flange} = 31.7$$

$$\lambda_{r\_flange} := \frac{238}{\sqrt{Fy}} \qquad \lambda_{r\_flange} = 39.7$$

$$\lambda_{flange} := \frac{b}{t_f} \qquad \lambda_{flange} = 2.5 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$P_u := |P_1|$ $P_y := Fy \cdot A$

$$\lambda_{p\_web} := \mathrm{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$\lambda_{p\_web} = 81$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$\lambda_{r\_web} = 151$

member values

$\lambda_{web} := \frac{h}{t_w}$ $\lambda_{web} = 8$ $\lambda_{web} < \lambda_{p\_web}$ and λ_{r_web}
OK

$$\lambda_c := \mathrm{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi}\cdot\sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi}\cdot\sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 0.63$

$$F_{cr} := \mathrm{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2}\cdot Fy, \frac{0.877}{\lambda_c^2}\cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 30.5$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \mathrm{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$

$\phi P_n = 337$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS	2 - 2 AXIS
$M_{p3\_initial} := Fy \cdot Z_3$	$M_{p2\_initial} := Fy \cdot Z_2$
$M_{y3} := Fy \cdot S_3$	$M_{y2} := Fy \cdot S_2$

$$M_{p3} := \mathrm{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$$

$$M_{p2} := \mathrm{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$$

$M_{p3} = 1089$ kip·in $\qquad$ $M_{p2} = 603$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS $\qquad$ 2 - 2 AXIS

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A} \qquad \text{(eq. LRFD F1-5)}$$

$$M_r := Fyf \cdot S_3$$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A} \qquad \text{(eq. LRFD F1-10)}$$

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right)\left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right] \qquad \text{(eq. LRFD F1-2)}$$

$M_{nLTB\_inelastic} = 1098$ kip·in

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2}\right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 1089 \quad kip \cdot in$$

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} = 1089 \quad kip \cdot in \qquad M_{n2} = 603 \quad kip \cdot in$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right)\left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right)\left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$$M_{n3} = 1089 \quad kip \cdot in$$

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\;2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$$B2_3 = 1.01$$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\;2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$$B2_2 = 1.03$$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 721$ kip·in $\qquad M_{u2} = 254$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 1.258$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 1.13$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf\cdot A_f$$

$$\phi V_{n3} := \phi_v\cdot V_{n3}$$

$\phi V_{n3} = 156$ kips

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \mathrm{if}\left(\lambda \le r1, 0.6\cdot Fyw\cdot A_w, 0\right)$$

$$V_{n2} := \mathrm{if}\left[(r1 \le \lambda)\cdot(\lambda \le r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \mathrm{if}\left[r2 \le \lambda, \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$$\phi V_{n2} := \phi_v\cdot V_{n2}$$

$\phi V_{n2} = 136$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$A_o := (b_f - t_f) \cdot (d - t_w)$$

$$f_{v3} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_3}{2 \cdot A_f} \qquad f_{v2} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_2}{2 \cdot A_w}$$

$$f_{v3} = 2.21 \text{ ksi} \qquad f_{v2} = 2.88 \text{ ksi}$$

$$\frac{f_{v3}}{F_v} = 0.114 \qquad \frac{f_{v2}}{F_v} = 0.148$$

Bending / Axial DCR controls

$$DCR := \begin{bmatrix} C1 \\ \dfrac{f_{v3}}{F_v} \\ \dfrac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 1.13$$

APPENDIX B6a

Type L Cab Upgraded Corner Mullion (Columns 1 - 4) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Type L, 30 ft. (San Carlos, CA)

MEMBER : retrofit mullion @ critical section

ANALYSIS RUN : L 12

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

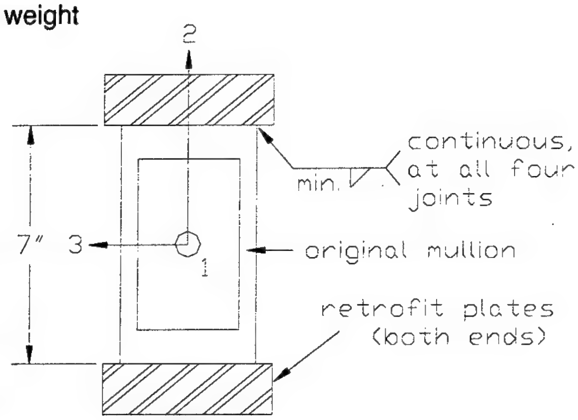
P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -21.6\,\text{kips}$ $\quad M_1 := 20\ \text{kip}\cdot\text{in}$

$V_2 := 27.4\ \text{kips}$ $\quad M_2 := 805\ \text{kip}\cdot\text{in}$

$V_3 := 18.5\ \text{kips}$ $\quad M_3 := 2722\ \text{kip}\cdot\text{in}$



$P_2 := |V_2|$

$P_3 := |V_3|$ $\quad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$t_p := 1.5$ in $\qquad k_{nt3} := 0.8$ $\qquad$ K values by LRFD Table C-C2.1

$b_p := 5$ in $\qquad k_{nt2} := 0.8$

$d := 7 + 2 \cdot t_p$ in $\qquad k_{lt3} := 2.10$

$t_w := 0.5$ in $\qquad k_{lt2} := 2.10$

$b_f := 4.0$ in $\qquad L_{b3} := 37.9$ in

$t_f := 1.0$ in $\qquad L_{b2} := 37.9$ in

MATERIAL & CODE PROPERTIES

$E := 29000$ ksi $\qquad Fy := 36$ ksi $\qquad \phi_b := 0.90$ $\qquad \phi_c := 0.85$

$G := 11200$ ksi $\qquad Fu := 58$ ksi $\qquad \phi_v := 0.90$ $\qquad C_b := 1.0$

$Fr := 10$ ksi $\qquad Fyf := Fy$ $\qquad \phi_{ty} := 0.90$

$F_L := Fyf - Fr$ $\qquad Fyw := Fy$ $\qquad \phi_{tf} := 0.75$

CALCULATED PROPERTIES

$$A := 2 \cdot b_p \cdot t_p + 2 \cdot b_f \cdot t_f + 2 \cdot \left(d - 2 \cdot t_f - 2 \cdot t_p\right) \cdot in^2$$

$A = 28 \quad in^2$

$A_e := A$

$$I_3 := \frac{1}{12} \cdot b_f \cdot \left(d - 2 \cdot t_p\right)^3 - \frac{1}{12} \cdot \left(b_f - 2 \cdot t_w\right) \cdot \left(d - 2 \cdot t_p - 2 \cdot t_f\right)^3 + \frac{2}{12} \cdot b_p \cdot t_p^{\,3} + 2 \cdot b_p \cdot t_p \cdot \left(\frac{d}{2} - \frac{t_p}{2}\right)$$

$I_3 = 357 \quad in^4$

$$I_2 := \left[\frac{1}{12} \cdot \left(d - 2 \cdot t_p\right) \cdot b_f^{\,3}\right] - \left[\frac{1}{12} \cdot \left(d - 2 \cdot t_p - 2 \cdot t_f\right) \cdot \left(b_f - 2 \cdot t_w\right)^3\right] + \frac{2}{12} \cdot t_p \cdot b_p^{\,3}$$

$I_2 = 57.3 \quad in^4$

$$S_3 := \frac{I_3}{\frac{d}{2}} \; in^3 \qquad S_2 := \frac{I_2}{\frac{b_p}{2}} \; in^3$$

$$Z_3 := \left[\left(b_f \cdot t_f\right) \cdot \left(\frac{d}{2} - \frac{t_f}{2} - t_p\right) + 2 \cdot \left[t_w \cdot \left(\frac{d}{2} - t_f - t_p\right) \right] \cdot \frac{\left(\frac{d}{2} - t_f - t_p\right)}{2} + b_p \cdot t_p \cdot \left(\frac{d}{2} - \frac{t_p}{2}\right) \right] \cdot 2$$

$Z_3 = 94$ $\quad in^3$

$$Z_2 := \left[\left(d - 2 \cdot t_f - 2 \cdot t_p\right) \cdot t_w \cdot \left(\frac{b_f}{2} - \frac{t_w}{2}\right) + \left(\frac{b_f}{2} \cdot t_f \cdot \frac{b_f}{4}\right) \cdot 2 + \left(\frac{b_p}{2} \cdot t_p \cdot \frac{b_p}{4}\right) \cdot 2 \right] \cdot 2$$

$Z_2 = 35.5$ $\quad in^3$

$$J := \frac{4 \cdot \left[\left(d - t_f - t_p\right) \cdot \left(b_f - t_w\right) \right]^2}{2 \cdot \left[\frac{\left(d - t_f - t_p\right)}{t_w} + \frac{b_f - t_w}{t_f + t_p} \right]} \qquad J = 84 \quad in^4$$

$r_3 := \sqrt{\frac{I_3}{A}}$ in $\qquad A_w := 2 \cdot d \cdot t_w$

$\qquad A_f := 2 \cdot b_f \cdot t_f + 2 \cdot b_p \cdot t_p$

$\qquad h := d - 3 \cdot t_f - 2 \cdot t_p$

$r_2 := \sqrt{\frac{I_2}{A}}$ in $\qquad b := b_f - 3 \cdot t_w$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A \right]$$ (eq. LRFD D1-1) (eq. LRFD D1-2)

$\phi P_{nt} = 907$ $\quad$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}}$ $\qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$

$\lambda_{p\_flange} = 31.7$ $\qquad \lambda_{r\_flange} = 39.7$

$\lambda_{flange} := \frac{b_p}{t_f + t_p}$ $\qquad \lambda_{flange} = 2$ $\qquad \lambda_{flange} < \lambda_{p\_flange}$ OK

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$\lambda_{p\_web} = 99.7$ $\lambda_{r\_web} = 159$

member values

$\lambda_{web} := \frac{h}{t_w}$ $\lambda_{web} = 8$ $\lambda_{web} < \lambda_{p\_web}$ OK

$$\lambda_c := \mathrm{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 0.624$

$$F_{cr} := \mathrm{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 30.6$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \mathrm{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$ $\phi P_n = 728$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS	2 - 2 AXIS
$M_{p3\_initial} := Fy \cdot Z_3$	$M_{p2\_initial} := Fy \cdot Z_2$
$M_{y3} := Fy \cdot S_3$	$M_{y2} := Fy \cdot S_2$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3 \right.$ $M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S\right.$

$M_{p3} = 3384$ kip·in $M_{p2} = 1238$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS	2 - 2 AXIS

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-5)

$$M_r := Fyf \cdot S_3$$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-10)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

$M_{nLTB\_inelastic} = 3406$ kip·in

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS | 2 - 2 AXIS

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2}\right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := \mathrm{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 3384 \quad kip \cdot in$$

$$M_{n3} := \mathrm{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \mathrm{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} := \mathrm{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n2} = 1238 \quad kip \cdot in$$

$$M_{n3} = 3384 \quad kip \cdot in$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \mathrm{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \mathrm{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \mathrm{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \mathrm{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$$M_{n3} = 3384 \quad kip \cdot in$$

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$M_{nt} := 0 \qquad B1 := 1$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$M_{lt3} := M_3$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\;2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1$

2 - 2 AXIS

$M_{lt2} := M_2$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\;2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 1.01$

$M_{u3} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$

$M_{u2} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$

$M_{u3} = 2726 \quad \mathrm{kip \cdot in} \qquad M_{u2} = 812 \quad \mathrm{kip \cdot in}$

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$P_u := |P_1|$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 1.64$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2 \cdot \frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 1.47$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

$$\phi V_{n3} := \phi_v \cdot V_{n3}$$

$\phi V_{n3} = 447$ kips

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \mathrm{if}\left(\lambda \le r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \mathrm{if}\left[(r1 \le \lambda) \cdot (\lambda \le r2), 0.6 \cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \mathrm{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$$\phi V_{n2} := \phi_v \cdot V_{n2}$$

$\phi V_{n2} = 194$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$A_o := \left(b_f - t_f\right) \cdot \left(d - t_w\right)$$

$$f_{v3} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_3}{2 \cdot A_f}$$

$$f_{v3} = 0.753 \quad \text{ksi}$$

$$\frac{f_{v3}}{F_v} = 0.0387$$

$$f_{v2} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_2}{2 \cdot A_w}$$

$$f_{v2} = 1.72 \quad \text{ksi}$$

$$\frac{f_{v2}}{F_v} = 0.0885$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \dfrac{f_{v3}}{F_v} \\ \dfrac{f_{v2}}{F_v} \end{bmatrix}$$

$$\max(DCR) = 1.47$$

APPENDIX B7

Type L Cab Corner Mullion-to-Shaft Roof Beam Connection Evaluation

(based on AISC-LRFD. 2nd ed., 1994)

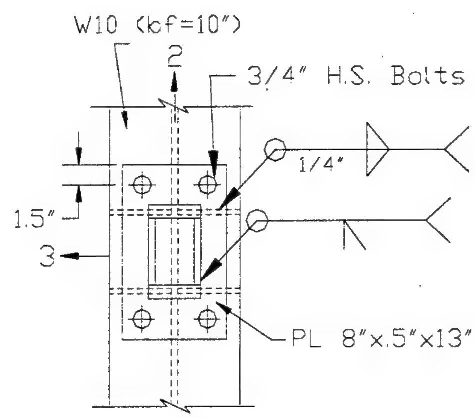
NOTES : -This sheet checks the strength of the base of the mullion connection for cab mullions 1-4.

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : Prismatic mullion base connection (dwg. S-4)

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight



MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 4.5$ kips $\quad M_1 := -70$ kip·in

$V_2 := 24.3$ kips $\quad M_2 := 66$ kip·in

$V_3 := 10.3$ kips $\quad M_3 := 804$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3| \qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

SECTION PROPERTIES

$d_b := 0.75$ in $\quad d := 7$ in

$t_p := 0.5$ in $\quad b := 4$ in

$l_p := 13$ in $\quad t_f := 1$ in

$t_w := 0.5$ in

$w_p := 8$ in $\quad arm := \frac{1}{2}\cdot(l_p - d + t_f)$

$d_{edge} := 1.5$ in

MATERIAL & CODE PROPERTIES

$F_{u\_b} := 90$ ksi $\quad \phi_v := 0.75$

$F_{y\_b} := 48$ ksi $\quad \phi_y := 0.90$

$F_w := 70$ ksi $\quad \phi_b := 0.75$

$F_y := 36$ ksi

CALCULATED PROPERTIES

$$A_b := \frac{\pi}{4} \cdot d_b^{\,2}$$

BENDING:

Maximum Moment may be limited by ultimate bolt tension (2 bolts):

$R_n := F_{u\_b} \cdot A_b \cdot 2 \qquad \phi_b \cdot R_n = 59.6$ kips

End Plate Connection (LRFD p. 10-24)

3-3 Axis Bending:

Values:

$$p_f := \frac{l_p}{2} - d_{edge} - \frac{d}{2}$$

$$p_e := p_f - \frac{d_b}{4}$$

$$b_f := b$$

$$b_p := w_p$$

$C_a := 1.36$ (Table 10-1)

$$C_b := \left(\frac{b_f}{b_p}\right)^{0.5}$$

$$A_f := b_f \cdot t_f$$

$$A_w := \left(d - 2 \cdot t_f\right) \cdot t_w \cdot 2$$

$$\alpha_m := C_a \cdot C_b \cdot \left(\frac{A_f}{A_w}\right)^{\frac{1}{3}} \cdot \left(\frac{p_e}{d_b}\right)^{\frac{1}{4}} \qquad \alpha_m = 1.03$$

Plate Bending Capacity:

$$M_n := \frac{b_p \cdot t_p^2}{4} \cdot F_y \qquad \phi_y \cdot M_n = 16.2 \quad \text{kip·in}$$

$$M_{en} := \phi_y \cdot M_n$$

$$M_t := \frac{M_{en}}{\alpha_m} \qquad M_t = 15.8 \quad \text{kip·in}$$

$$P_{uf} := M_t \cdot \left(\frac{2}{p_e}\right) \qquad P_{uf} = 24 \quad \text{kips} \quad < \phi_b \cdot R_n \text{ above}$$

$$\phi M_{u3} := \left(P_{uf} - \frac{P_1}{2}\right) \cdot \left(d - t_f\right) \qquad \phi M_{u3} = 131 \quad \text{kip·in}$$

2-2 Axis Bending:

Values:

$$p_f := \frac{w_p}{2} - d_{edge} - \frac{b}{2}$$

$$p_e := p_f - \frac{d_b}{4}$$

$$b_f := d$$

$$b_p := d + 1$$

$$C_a := 1.36 \qquad \text{(Table 10-1)}$$

$$C_b := \left(\frac{b_f}{b_p}\right)^{0.5}$$

$$A_f := b_f \cdot t_w$$

$$A_w := \left(b - 2 \cdot t_w\right) \cdot t_f \cdot 2$$

$$\alpha_m := C_a \cdot C_b \cdot \left(\frac{A_f}{A_w}\right)^{\frac{1}{3}} \cdot \left(\frac{p_e}{d_b}\right)^{\frac{1}{4}} \qquad \alpha_m = 0.854$$

Plate Bending Capacity:

$$M_n := \frac{b_p \cdot t_p^2}{4} \cdot F_y \qquad \phi_y \cdot M_n = 16.2 \quad \text{kip·in}$$

$$M_{en} := \phi_y \cdot M_n$$

$M_t := \dfrac{M_{en}}{\alpha_m}$ $\quad M_t = 19$ kip·in

$P_{uf} := M_t \cdot \left(\dfrac{2}{p_e}\right)$ $\quad P_{uf} = 121$ kips $\quad > \phi_b \cdot R_n$ above

Maximum tension of bolts controls:

$P_{uf} := \phi_b \cdot R_n$ $\quad P_{uf} = 59.6$ kips

$\phi M_{u2} := \left(P_{uf} - \dfrac{P_1}{2}\right) \cdot (b - t_w)$ $\quad \phi M_{u2} = 201$ kip·in

SHEAR :

Shear on bolts (limits shear reactions):

- 4 bolts total

$R_{n\_b} := A_b \cdot F_{y\_b} \cdot 4$ $\quad \phi_v \cdot R_{n\_b} = 63.6$ kips

Shear on plate (limits axial tension):

$R_{n\_pl3} := 0.60 \cdot w_p \cdot t_p \cdot F_y$ $\quad \phi_y \cdot R_{n\_pl3} = 77.8$ kips $\quad > \phi_b \cdot R_n$ above

$R_{n\_pl2} := 0.60 \cdot (d + 1) \cdot t_p \cdot F_y$ $\quad \phi_y \cdot R_{n\_pl2} = 77.8$ kips $\quad > \phi_b \cdot R_n$ above

Both plate shear forces are larger than maximum bolt tension: Bolts control

DEMAND CAPACITY RATIO (DCR):

Bending: Not additive in the 3 and 2 directions because plate bending controls in the 3 direction while the bolt tension controls in the 2 direction.

$$DCR := \begin{bmatrix} \dfrac{M_3}{\phi M_{u3}} \\ \dfrac{M_2}{\phi M_{u2}} \\ \dfrac{P_2}{\phi_v \cdot R_{n\_b}} \end{bmatrix}$$

$\max(DCR) = 6.15$

APPENDIX B8

Type L Cab Corner Mullion-to-Shaft Roof Beam Connection (Col 5) Evaluation

(based on AISC-LRFD. 2nd ed., 1994)

NOTES : -This sheet checks the strength of the base of the mullion connection for cab mullion 5.

TOWER : Type L, 50 ft. (Salinas, CA)

CONNECTION : Non-prismatic mullion base connection (dwg. S-4)

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

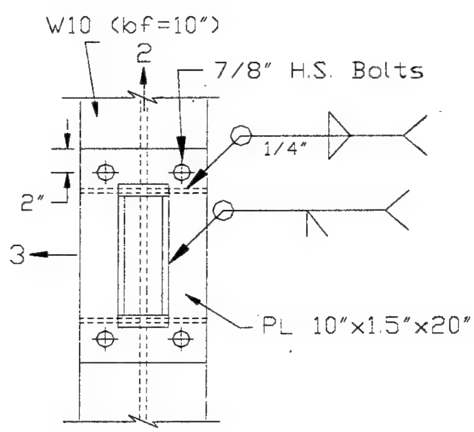
P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 23.5$ kips $\quad M_1 := -48$ kip·in

$V_2 := 26.5$ kips $\quad M_2 := -410$ kip·in

$V_3 := -15.0$ kips $\quad M_3 := -294$ kip·in



$P_2 := |V_2|$

$P_3 := |V_3| \qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

SECTION PROPERTIES

$d_b := 0.875$ in $\quad d := 12$ in

$t_p := 1.5$ in $\quad b := 4$ in

$l_p := 20$ in $\quad t_f := 1$ in

$w_p := 8$ in $\quad t_w := 0.5$ in

$d_{edge} := 2$ in $\quad arm := \frac{1}{2}\cdot(l_p - d + t_f)$

MATERIAL & CODE PROPERTIES

$F_{u\_b} := 90$ ksi $\qquad \phi_v := 0.75$

$F_{y\_b} := 48$ ksi $\qquad \phi_y := 0.90$

$F_w := 70$ ksi $\qquad \phi_b := 0.75$

$F_y := 36$ ksi

CALCULATED PROPERTIES

$$A_b := \frac{\pi}{4} \cdot d_b^{\,2}$$

BENDING:

Maximum Moment may be limited by ultimate bolt tension (2 bolts):

$R_n := F_{u\_b} \cdot A_b \cdot 2 \qquad \phi_b \cdot R_n = 81.2$ kips

End Plate Connection (LRFD p. 10-24)

3-3 Axis Bending:

Values:

$$p_f := \frac{l_p}{2} - d_{edge} - \frac{d}{2}$$

$$p_e := p_f - \frac{d_b}{4}$$

$$b_f := b$$

$$b_p := b_f + 1$$

$C_a := 1.36$ (Table 10-1)

$$C_b := \left(\frac{b_f}{b_p}\right)^{0.5}$$

$$A_f := b_f \cdot t_f$$

$$A_w := \left(d - 2 \cdot t_f\right) \cdot t_w \cdot 2$$

$$\alpha_m := C_a \cdot C_b \cdot \left(\frac{A_f}{A_w}\right)^{\frac{1}{3}} \cdot \left(\frac{p_e}{d_b}\right)^{\frac{1}{4}} \qquad \alpha_m = 1.07$$

Plate Bending Capacity:

$M_n := \frac{b_p \cdot t_p^2}{4} \cdot F_y$ $\quad \phi'_y \cdot M_n = 91.1$ kip·in

$M_{en} := \phi_y \cdot M_n$

$M_t := \frac{M_{en}}{\alpha_m}$ $\quad M_t = 85.1$ kip·in

$P_{uf} := M_t \cdot \left(\frac{2}{p_e}\right)$ $\quad P_{uf} = 95.6$ kips > $\phi_b \cdot R_n$ above

Maximum tension of bolts controls:

$P_{uf} := \phi_b \cdot R_n$ $\quad P_{uf} = 81.2$ kips

$\phi M_{u3} := \left(P_{uf} - \frac{P_1}{2}\right) \cdot (d - t_f)$ $\quad \phi M_{u3} = 764$ kip·in

2-2 Axis Bending:

Values:

$p_f := 3 - 2$

$p_e := p_f - \frac{d_b}{4}$

$b_f := d$

$b_p := d + 1$

$C_a := 1.36$ (Table 10-1)

$C_b := \left(\frac{b_f}{b_p}\right)^{0.5}$

$A_f := b_f \cdot t_w$

$A_w := (b - 2 \cdot t_w) \cdot t_f \cdot 2$

$\alpha_m := C_a \cdot C_b \cdot \left(\frac{A_f}{A_w}\right)^{\frac{1}{3}} \cdot \left(\frac{p_e}{d_b}\right)^{\frac{1}{4}}$ $\quad \alpha_m = 1.27$

Plate Bending Capacity:

$M_n := \frac{b_p \cdot t_p^2}{4} \cdot F_y$ $\quad \phi_y \cdot M_n = 237$ kip·in

$M_{en} := \phi_y \cdot M_n$

$M_t := \dfrac{M_{en}}{\alpha_m}$ $M_t = 187$ kip·in

$P_{uf} := M_t \cdot \left(\dfrac{2}{p_e}\right)$ $P_{uf} = 478$ kips $> \phi_b \cdot R_n$ above

Maximum tension of bolts controls:

$P_{uf} := \phi_b \cdot R_n$ $P_{uf} = 81.2$ kips

$\phi M_{u2} := \left(P_{uf} - \dfrac{P_1}{2}\right) \cdot (b - t_w)$ $\phi M_{u2} = 243$ kip·in

SHEAR :

Shear on bolts (limits shear reactions):

- 4 bolts total

$R_{n\_b} := A_b \cdot F_{y\_b} \cdot 4$ $\phi_v \cdot R_{n\_b} = 86.6$ kips

Shear on plate (limits axial tension):

$R_{n\_pl3} := 0.60 \cdot (b + 1) \cdot t_p \cdot F_y$ $\phi_y \cdot R_{n\_pl3} = 146$ kips $> \phi_b \cdot R_n$ above

$R_{n\_pl2} := 0.60 \cdot (d + 1) \cdot t_p \cdot F_y$ $\phi_y \cdot R_{n\_pl2} = 379$ kips $> \phi_b \cdot R_n$ above

Both plate shear forces are larger than maximum bolt tension: Bolts control

INTERACTION:

Bending: Additive in the 3 and 2 directions as the bolt tension controls in both directions.

$$DCR := \begin{bmatrix} \dfrac{M_3}{\phi M_{u3}} + \dfrac{M_2}{\phi M_{u2}} \\ \dfrac{P_2}{\phi_v \cdot R_{n\_b}} \end{bmatrix}$$

$\max(DCR) = 2.07$

APPENDIX B9

Type L 10WF72 Where it Crosses the 16WF88 Shaft Roof Beam Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for I-shaped members
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : 10WF72 @ 16WF88 / shaft roof

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

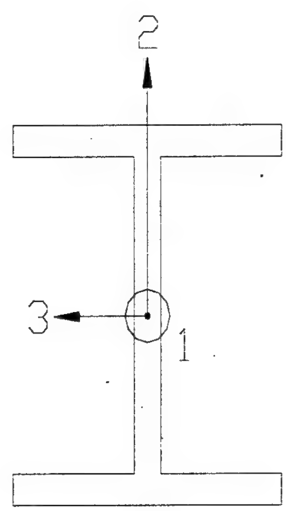
P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -29.7\ \text{kips}$ $M_1 := 22\ \text{kip}\cdot\text{in}$

$V_2 := 56.6\ \text{kips}$ $M_2 := -233\ \text{kip}\cdot\text{in}$

$V_3 := 4.8\ \text{kips}$ $M_3 := -3101\ \text{kip}\cdot\text{in}$



the 1 axis is the longitudinal axis out of the page

$P_2 := |V_2|$

$P_3 := |V_3|$

$M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 21.18\ in^2$	$J := 3.56\ in^4$	$k_{nt3} := 2$	
$A_e := A$	$d := 10.50\ in$	$k_{nt2} := 2$	K values by LRFD Table C-C2.1
$I_3 := 420.7\ in^4$	$k := 1 + \frac{5}{16}\ in$	$k_{lt3} := 2$	
$I_2 := 141.8\ in^4$	$t_w := 0.510\ in$	$k_{lt2} := 2$	
$S_3 := 80.1\ in^3$	$b_f := 10.170\ in$	$L_{b3} := 43.4\ in$	
$S_2 := 27.9\ in^3$	$t_f := 0.808\ in$	$L_{b2} := 43.4\ in$	
$Z_3 := 85.3\ in^3$	$C_w := 3100\ in^6$		
$Z_2 := 40.1\ in^3$			

MATERIAL & CODE PROPERTIES

$E := 29000\ ksi$	$Fy := 36\ ksi$	$\phi_b := 0.90$	$\phi_c := 0.85$
$G := 11200\ ksi$	$Fu := 58\ ksi$	$\phi_v := 0.90$	$C_b := 1.0$
$Fr := 10\ ksi$	$Fyf := Fy$	$\phi_{ty} := 0.90$	C_b is conservatively taken equal to 1.0
$F_L := Fyf - Fr$	$Fyw := Fy$	$\phi_{tf} := 0.75$	

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 2 \cdot k$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, \left(\phi_{tf}\cdot Fu\cdot A_e\right), \phi_{ty}\cdot Fy\cdot A\right]$$

(eq. LRFD D1-1)
(eq. LRFD D1-2)

$\phi P_{nt} = 686$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{p\_flange} = 10.8$$

$$\lambda_{r\_flange} := \frac{141}{\sqrt{Fy - 10}} \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2\cdot t_f} \qquad \lambda_{flange} = 6.29 \qquad \lambda_{flange} < \lambda_{p\_flange} \text{ OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := \left| P_1 \right|$$

$$P_y := Fy\cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b\cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b\cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75\cdot P_u}{\phi_b\cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{p\_web} = 94$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74\cdot\frac{P_u}{\phi_b\cdot P_y}\right) \qquad \lambda_{r\_web} = 156$$

member values $\lambda_{web} := \dfrac{h}{t_w}$ $\qquad \lambda_{web} = 15.4$

$\lambda_{web} < \lambda_{p\_web}$ OK

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 0.376$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 33.9$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$ $\phi P_n = 611$ kips

---

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 3071$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 1444$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L{}^2}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}}\right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 3071 \quad \text{kip} \cdot \text{in}$$

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$ $$M_{n2} := M_{p2}$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$M_{n3} := \mathrm{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$

$M_{n3} = 3071$ kip·in $\qquad M_{n2} = 1444$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right] \qquad \text{(eq. LRFD A-F1-3)}$$

$M_{nFLB} := \mathrm{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right] \qquad \text{(eq. LRFD A-F1-3)}$$

$M_{nWLB} := \mathrm{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$

$M_{n3} := \mathrm{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$

$M_{n3} := \mathrm{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$

$M_{n3} = 3071$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$M_{nt} := 0$

$B1 := 1$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$M_{lt3} := M_3$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{2}}$$

2 - 2 AXIS

$M_{lt2} := M_2$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}} \qquad B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$$B2_3 = 1 \qquad B2_2 = 1$$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 3107$ kip·in $\qquad M_{u2} = 234$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 1.3$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2 \cdot \frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 1.2$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u> <u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w} \qquad \lambda = 15.4$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

Flanges will not buckle due to shear as in the web. Therefore:

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

$$V_{n2} := \text{if}\left(\lambda \le r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \le \lambda) \cdot (\lambda \le r2), 0.6 \cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 355$ kips $\qquad V_{n2} = 116$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3} \qquad \phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 319$ kips $\qquad \phi V_{n2} = 104$ kips

$V_3 = 4.8$ kips $\qquad V_2 = 56.6$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{2 \cdot A_f} \qquad f_{v2} := \frac{P_2}{A_w}$$

$f_{v3} = 0.401$ ksi $\qquad f_{v2} = 10.6$ ksi

$$\frac{f_{v3}}{F_v} = 0.0206 \qquad \frac{f_{v2}}{F_v} = 0.544$$

Bending / Axial DCR controls

$$DCR := \begin{bmatrix} C1 \\ \dfrac{f_{v3}}{F_v} \\ \dfrac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 1.19$$

APPENDIX B10

Type L 12WF27 Cab Floor Beam Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

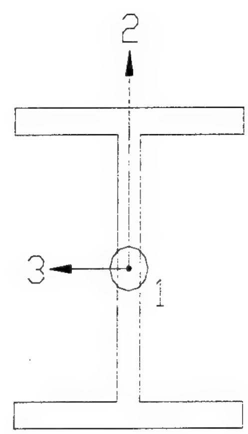
NOTES : -This sheet is for I-shaped members
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : 12WF27 Cab Floor near Col 5

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight



the 1 axis is the longitudinal axis out of the page

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 16.8$ kips $\qquad M_1 := 0$ kip·in

$V_2 := -5.8$ kips $\qquad M_2 := -79$ kip·in

$V_3 := 1.3$ kips $\qquad M_3 := 390$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$

$M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 7.97 \ in^2$	$J := 0.30 \ in^4$	$k_{nt3} := 1$	
$A_e := A$	$d := 11.96 \ in$	$k_{nt2} := 1$	K values by LRFD Table C-C2.1
$I_3 := 204.1 \ in^4$	$k := 0 + \frac{13}{16} \ in$	$k_{lt3} := 1$	
$I_2 := 16.6 \ in^4$	$t_w := 0.240 \ in$	$k_{lt2} := 1$	
$S_3 := 34.1 \ in^3$	$b_f := 6.50 \ in$	$L_{b3} := 192 \ in$	
$S_2 := 5.1 \ in^3$	$t_f := 0.400 \ in$	$L_{b2} := 129 \ in$	
$Z_3 := 37.2 \ in^3$	$C_w := 607 \ in^6$		
$Z_2 := 8.17 \ in^3$			

MATERIAL & CODE PROPERTIES

$E := 29000 \ ksi$	$Fy := 36 \ ksi$	$\phi_b := 0.90$	$\phi_c := 0.85$
$G := 11200 \ ksi$	$Fu := 58 \ ksi$	$\phi_v := 0.90$	$C_b := 1.0$
$Fr := 10 \ ksi$	$Fyf := Fy$	$\phi_{ty} := 0.90$	C_b is conservatively taken equal to 1.0
$F_L := Fyf - Fr$	$Fyw := Fy$	$\phi_{tf} := 0.75$	

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 2 \cdot k$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)

(eq. LRFD D1-2)

$\phi P_{nt} = 258$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy - 10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2 \cdot t_f} \qquad \lambda_{flange} = 8.13 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \geq \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \leq 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 87.6 \qquad \lambda_{r\_web} = 154$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 43.1 \qquad \lambda_{web} < \lambda_{p\_web} \quad \text{OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right) \quad \text{(eq.LRFD E2-4)}$$

$$\lambda_c = 1$$

$$F_{cr} := \text{if}\left(\lambda_c \leq 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right) \quad \text{(eq.LRFD E2-2)} \quad \text{(eq.LRFD E2-3)}$$

$F_{cr} = 23.6$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$

$\phi P_n = 258$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 1339$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 275$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

2 - 2 AXIS

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^{\,2}}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}} \right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 1339 \quad \text{kip} \cdot \text{in}$$

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0 \right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} = 946 \quad \text{kip} \cdot \text{in}$$

$$M_{n2} = 275 \quad \text{kip} \cdot \text{in}$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange} \right) \cdot \left(M_{nFLB} < M_{p3} \right), M_{nFLB}, M_{p3} \right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3} \right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 946$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^2}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1.01$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^2}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 1.06$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 390$ kip·in

$M_{u2} = 79$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.809$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.727$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u>

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

<u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \mathrm{if}\left(\lambda \leq r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \mathrm{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6 \cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 112$ kips

$V_{n2} = 62$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3}$

$\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 101$ kips

$\phi V_{n2} = 55.8$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6 \cdot \phi_v \cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{2 \cdot A_f}$$

$$f_{v2} := \frac{P_2}{A_w}$$

$f_{v3} = 0.125$ ksi

$f_{v2} = 2.02$ ksi

$\frac{f_{v3}}{F_v} = 0.00643$

$\frac{f_{v2}}{F_v} = 0.104$

Bending / Axial DCR controls

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

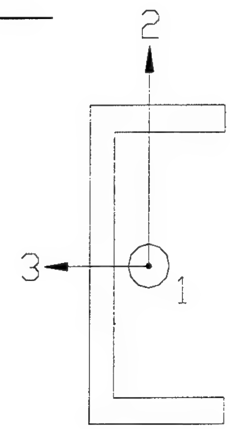
$\max(DCR) = 0.727$

APPENDIX B11

Type L Channel at the Base of the Cab Window (C4 x 7.25) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for C-shaped members
-Note member axis definitions below



the 1 axis is the longitudinal axis out of the page

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : C4x7.25 at base of window

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -19.3$ kips $\quad M_1 := 0$ kip·in

$V_2 := 2.5$ kips $\quad M_2 := 0$ kip·in

$V_3 := -0.9$ kips $\quad M_3 := 123$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$

$M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 2.13 \ in^2$

$A_e := A$

$I_3 := 4.59 \ in^4$

$I_2 := 0.433 \ in^4$

$S_3 := 2.29 \ in^3$

$S_2 := 0.343 \ in^3$

$Z_3 := 2.81 \ in^3$

$Z_2 := 0.697 \ in^3$

$d := 4.00 \ in$

$k := 0 + \frac{11}{16} \ in$

$t_w := 0.321 \ in$

$b_f := 1.721 \ in$

$t_f := 0.296 \ in$

$x_o := 0.459 \ in$

$e_o := 0.386 \ in$

$k_{nt3} := 1$

$k_{nt2} := 1$

$k_{lt3} := 1.0$

$k_{lt2} := 1.0$

K values by LRFD Table C-C2.1

$k_1 := 1.0$ torsional effective length factor

$L_{b3} := 192 \ in$

$L_{b2} := 12 \ in$

$L_{b1} := 192 \ in$ unbraced length for torsion

MATERIAL & CODE PROPERTIES

$E := 29000 \ ksi$

$G := 11200 \ ksi$

$Fr := 10 \ ksi$

$F_L := Fyf - Fr$

$Fy := 36 \ ksi$

$Fu := 58 \ ksi$

$Fyf := Fy$

$Fyw := Fy$

$\phi_b := 0.90$

$\phi_v := 0.90$

$\phi_{ty} := 0.90$

$\phi_{tf} := 0.75$

$\phi_c := 0.85$

$C_b := 1.0$

C_b is conservatively taken equal to 1.0

flexural-torsional properties (pg 1-145)

$C_w := 1.24 \ in^6$ warping constant

$J := 0.08 \ in^4$ torsional constant

$H := 0.768$ flexural constant

$r_o := 1.75 \ in$ polar radius of gyration about shear center

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}}$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

$A_w := d \cdot t_w$

$A_f := 2 \cdot b_f \cdot t_f$

$h := d - 2 \cdot k$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, \left(\phi_{tf}\cdot Fu\cdot A_e\right), \phi_{ty}\cdot Fy\cdot A\right]$$ (eq. LRFD D1-1) (eq. LRFD D1-2)

$\phi P_{nt} = 69$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy - 10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{t_f} \qquad \lambda_{flange} = 5.81 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := \left|P_1\right|$$

$$P_y := Fy\cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b\cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b\cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75\cdot P_u}{\phi_b\cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74\cdot\frac{P_u}{\phi_b\cdot P_y}\right)$$

$$\lambda_{p\_web} = 65.3 \qquad \lambda_{r\_web} = 128$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 8.18 \qquad \lambda_{web} < \lambda_{p\_web} \quad \text{OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 1.47$

$$F_{crFB} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2)

(eq.LRFD E2-3)

$F_{crFB} = 14.6$ ksi

CHECK FLEXURAL-TORSIONAL BUCKLING

$$F_{ey} := \frac{\pi^2 \cdot E}{\left(\frac{k_{lt3} \cdot L_{b3}}{r_3}\right)^2}$$ y represents axis of symmetry

$$F_{ez} := \frac{\left[\frac{\pi^2 \cdot E \cdot C_w}{\left(k_1 \cdot L_{b1}\right)^2} + G \cdot J\right]}{A \cdot r_o^2}$$

$$F_e := \frac{F_{ey} + F_{ez}}{2 \cdot H} \cdot \sqrt{1 - \frac{4 \cdot F_{ey} \cdot F_{ez} \cdot H}{\left(F_{ey} + F_{ez}\right)^2}}$$

$$\lambda_e := \sqrt{\frac{Fy}{F_e}}$$

$$F_{crTB} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$

$$F_{cr} := \text{if}\left(F_{crTB} < F_{crFB}, F_{crTB}, F_{crFB}\right)$$

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$$

$\phi P_n = 26.5$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$$M_{p3\_initial} := Fy \cdot Z_3$$

$$M_{y3} := Fy \cdot S_3$$

$$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$$

$M_{p3} = 101$ kip·in

2 - 2 AXIS

$$M_{p2\_initial} := Fy \cdot Z_2$$

$$M_{y2} := Fy \cdot S_2$$

$$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$$

$M_{p2} = 18.5$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^{\,2}}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - (M_{p3} - M_r) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS — 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}} \right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 101$ kip·in

$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$ — $M_{n2} := M_{p2}$

$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$

$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$

$M_{n3} = 101$ kip·in — $M_{n2} = 18.5$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - (M_{p3} - M_r) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - (M_{p3} - M_r) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right]$$ (eq. LRFD A-F1-3)

$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$

$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$

$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$

$M_{n3} = 101$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\ 2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$$B2_3 = 2.18$$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\ 2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$$B2_2 = 1.02$$

$$M_{u3} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 268$ kip·in $\qquad$ $M_{u2} = 0$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) $\qquad$ (eq. LRFD H1-1b)

$C = 3.35$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 2.98$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf\cdot A_f$$

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \text{if}\left(\lambda \leq r1, 0.6\cdot Fyw\cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \leq \lambda, \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 22$ kips | $V_{n2} = 27.7$ kips

$\phi V_{n3} := \phi_v\cdot V_{n3}$ | $\phi V_{n2} := \phi_v\cdot V_{n2}$

$\phi V_{n3} = 19.8$ kips | $\phi V_{n2} = 25$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60):

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1 + P_2 \cdot (e_o + x_o)}{A_f \cdot \frac{d}{2}} + \frac{P_3}{A_f} \qquad f_{v2} := \frac{P_2}{A_w}$$

$$f_{v3} = 1.92 \quad \text{ksi} \qquad f_{v2} = 1.95 \quad \text{ksi}$$

$$\frac{f_{v3}}{F_v} = 0.0988 \qquad \frac{f_{v2}}{F_v} = 0.1$$

Bending / Axial DCR controls

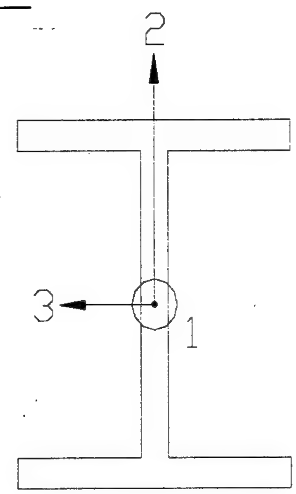
$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 2.98$$

APPENDIX B12

Type L Cab Interior Mullion (S3 x 7.5) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for I-shaped members
-Note member axis definitions below



TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : S3x7.5 Interior Mullion

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -16.0\,\text{kips}$ $\quad M_1 := 1.0\ \text{kip}\cdot\text{in}$

$V_2 := 0.0\ \text{kips}$ $\quad M_2 := 0\ \text{kip}\cdot\text{in}$

$V_3 := 0\ \text{kips}$ $\quad M_3 := 1\ \text{kip}\cdot\text{in}$

the 1 axis is the longitudinal axis out of the page

$P_2 := |V_2|$

$M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

$P_3 := |V_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.
-V3 and M2 set to zero due to the influence of the cab glass in the actual structure.

SECTION PROPERTIES

$A := 2.17$ in^2	$J := 0.09$ in^4	$k_{nt3} := 0.65$	
$A_e := A$	$d := 3.00$ in	$k_{nt2} := 1$	K values by LRFD Table C-C2.1
$I_3 := 2.9$ in^4	$k := 0 + \frac{9}{16}$ in	$k_{lt3} := 1.2$	
$I_2 := 0.59$ in^4	$t_w := 0.349$ in	$k_{lt2} := 1$	
$S_3 := 1.9$ in^3	$b_f := 2.509$ in	$L_{b3} := 90.4$ in	
$S_2 := 0.47$ in^3	$t_f := 0.260$ in	$L_{b2} := 90.4$ in	
$Z_3 := 2.36$ in^3	$C_w := 1.10$ in^6		
$Z_2 := 0.826$ in^3			

MATERIAL & CODE PROPERTIES

$E := 29000$ ksi	$Fy := 36$ ksi	$\phi_b := 0.90$	$\phi_c := 0.85$
$G := 11200$ ksi	$Fu := 58$ ksi	$\phi_v := 0.90$	$C_b := 1.0$
$Fr := 10$ ksi	$Fyf := Fy$	$\phi_{ty} := 0.90$	C_b is conservatively taken equal to 1.0
$F_L := Fyf - Fr$	$Fyw := Fy$	$\phi_{tf} := 0.75$	

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 2 \cdot k$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

(eq. LRFD D1-1)

$$\phi P_{nt} := \text{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-2)

$\phi P_{nt} = 70.3$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy - 10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2 \cdot t_f} \qquad \lambda_{flange} = 4.82 \qquad \lambda_{flange} < \lambda_{p\_flange} \text{ OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \geq \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \leq 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 66.9 \qquad \lambda_{r\_web} = 134$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 5.37 \qquad \lambda_{web} < \lambda_{p\_web} \text{ OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right) \qquad \text{(eq.LRFD E2-4)}$$

$$\lambda_c = 1.9$$

$$F_{cr} := \text{if}\left(\lambda_c \leq 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$

(eq.LRFD E2-2)

(eq.LRFD E2-3)

$$F_{cr} = 8.35 \text{ ksi}$$

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \mathrm{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$

$\phi P_n = 15.4$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS	2 - 2 AXIS
$M_{p3\_initial} := Fy \cdot Z_3$	$M_{p2\_initial} := Fy \cdot Z_2$
$M_{y3} := Fy \cdot S_3$	$M_{y2} := Fy \cdot S_2$

$M_{p3} := \mathrm{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p2} := \mathrm{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p3} = 85$ kip·in

$M_{p2} = 25.4$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS	2 - 2 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^2}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS | 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}} \right)^2 \cdot I_2 \cdot C_W}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \mathrm{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 85$ kip·in

$$M_{n3} := \mathrm{if}\left(L_{b2} < L_p, M_{p3}, 0 \right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \mathrm{if}\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := \mathrm{if}\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$M_{n3} = 75.4$ kip·in | $M_{n2} = 25.4$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \mathrm{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange} \right) \cdot \left(M_{nFLB} < M_{p3} \right), M_{nFLB}, M_{p3} \right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \mathrm{if}\left[(\lambda_{web} > \lambda_{p\_web})\cdot(M_{nWLB} < M_{p3}), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \mathrm{if}(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3})$$

$$M_{n3} := \mathrm{if}(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3})$$

$M_{n3} = 75.4$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt}\cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3}\cdot L_{b3}}{r_3\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A\cdot Fy}{\lambda_{c3}^2}$$

$$B2_3 := \frac{1}{1-\frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1.29$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2}\cdot L_{b2}}{r_2\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A\cdot Fy}{\lambda_{c2}^2}$$

$$B2_2 := \frac{1}{1-\frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 4.43$

$$M_{u3} := \mathrm{if}(P_1 > 0, M_{nt} + M_{lt3}, B1\cdot M_{nt} + B2_3\cdot M_{lt3})$$

$$M_{u2} := \mathrm{if}(P_1 > 0, M_{nt} + M_{lt2}, B1\cdot M_{nt} + B2_2\cdot M_{lt2})$$

$M_{u3} = 1.29$ kip·in $\qquad$ $M_{u2} = 0$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 1.06$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.898$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u> <u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

Flanges will not buckle due to shear as in the web. Therefore:

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n3} := 0.6\cdot Fyf\cdot A_f \qquad V_{n2} := \mathrm{if}(\lambda \leq r1, 0.6\cdot Fyw\cdot A_w, 0)$$

$$V_{n2} := \mathrm{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \mathrm{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2} \right]$$

$V_{n3} = 28.2$ kips

$V_{n2} = 22.6$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3}$

$\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 25.4$ kips

$\phi V_{n2} = 20.4$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6 \cdot \phi_v \cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{A_f}$$

$$f_{v2} := \frac{P_2}{A_w}$$

$f_{v3} = 0.511$ ksi

$f_{v2} = 0$ ksi

$$\frac{f_{v3}}{F_v} = 0.0263$$

$$\frac{f_{v2}}{F_v} = 0$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$\max(DCR) = 0.898$

APPENDIX B13

Type L Cab Corner Mullion at the Roof Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : Corner Mullion @ roof

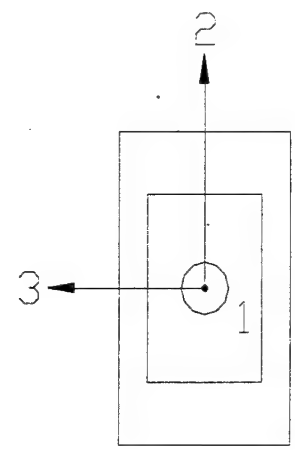
ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression



the 1 axis is the longitudinal axis out of the page

$P_1 := -4.9$ kips $M_1 := -50$ kip·in

$V_2 := 9.9$ kips $M_2 := -150$ kip·in

$V_3 := -5.3$ kips $M_3 := 266$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$ $M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$d := 7$ in

$t_w := 0.5$ in

$b_f := 4.0$ in

$t_f := 1.0$ in

$J := 50$ in^4

$k_{nt3} := 0.8$

$k_{nt2} := 0.8$

$k_{lt3} := 2.10$

$k_{lt2} := 2.10$

K values by LRFD Table C-C2.1

$L_{b3} := 26.7$ in

$L_{b2} := 26.7$ in

MATERIAL & CODE PROPERTIES

$E := 29000$ ksi

$G := 11200$ ksi

$Fr := 10$ ksi

$F_L := Fyf - Fr$

$Fy := 36$ ksi

$Fu := 58$ ksi

$Fyf := Fy$

$Fyw := Fy$

$\phi_b := 0.90$

$\phi_v := 0.90$

$\phi_{ty} := 0.90$

$\phi_{tf} := 0.75$

$\phi_c := 0.85$

$C_b := 1.0$

CALCULATED PROPERTIES

$A := \left[b_f \cdot t_f + \left(d - 2 \cdot t_f\right) \cdot t_w\right] \cdot 2$

$A = 13$ in^2

$A_e := A$

$$I_3 := b_f \cdot \frac{d^3}{12} - \frac{\left(b_f - 2 \cdot t_w\right) \cdot \left(d - 2 \cdot t_f\right)^3}{12} \qquad I_3 = 83.1 \quad \text{in}^4$$

$$I_2 := \frac{d \cdot b_f^{\,3}}{12} - \frac{\left(b_f - 2 \cdot t_w\right)^3 \cdot \left(d - 2 \cdot t_f\right)}{12} \qquad I_2 = 26.1 \quad \text{in}^4$$

$$S_3 := \frac{I_3}{\left(\frac{d}{2}\right)} \qquad S_3 = 23.7 \quad \text{in}^3$$

$$S_2 := \frac{I_2}{\left(\frac{b_f}{2}\right)} \qquad S_2 = 13 \quad \text{in}^3$$

$$Z_3 := \left[(b_f \cdot t_f) \cdot \left(\frac{d}{2} - \frac{t_f}{2} \right) + 2 \cdot \left[t_w \cdot \left(\frac{d}{2} - t_f \right) \right] \cdot \frac{\left(\frac{d}{2} - t_f \right)}{2} \right] \cdot 2 \qquad Z_3 = 30.3 \quad \text{in}^3$$

$$Z_2 := \left[(d - 2 \cdot t_f) \cdot t_w \cdot \left(\frac{b_f}{2} - \frac{t_w}{2} \right) + \frac{b_f}{2} \cdot t_f \cdot \frac{b_f}{4} \cdot 2 \right] \cdot 2 \qquad Z_2 = 16.8 \quad \text{in}^3$$

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := 2 \cdot d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 3 \cdot t_f$$

$$r_2 := \sqrt{\frac{I_2}{A}} \qquad b := b_f - 3 \cdot t_w$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e \right), \phi_{ty} \cdot Fy \cdot A \right]$$ (eq. LRFD D1-1) (eq. LRFD D1-2)

$\phi P_{nt} = 421$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$$

$$\lambda_{p\_flange} = 31.7 \qquad \lambda_{r\_flange} = 39.7$$

$$\lambda_{flange} := \frac{b}{2 \cdot t_f} \qquad \lambda_{flange} = 1.25 \qquad \lambda_{flange} < \lambda_{p\_flange} \ \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125 \right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y} \right) \ge \frac{253}{\sqrt{Fy}} \right] \right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y} \right), \frac{253}{\sqrt{Fy}} \right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$\lambda_{p\_web} = 103$ $\lambda_{r\_web} = 160$

member values

$\lambda_{web} := \frac{h}{t_w}$ $\lambda_{web} = 8$ $\lambda_{web} < \lambda_{p\_web}$ OK

$$\lambda_c := \mathrm{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 0.444$

$$F_{cr} := \mathrm{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 33.1$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \mathrm{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$ $\phi P_n = 366$ kips

---

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y3} := Fy \cdot S_3$

$M_{y2} := Fy \cdot S_2$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p3} = 1089$ kip·in

$M_{p2} = 603$ kip·in

(a) : rectangular bars and box sections

<u>3 - 3 AXIS</u>

<u>2 - 2 AXIS</u>

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-5)

$M_r := Fyf \cdot S_3$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-10)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

$M_{nLTB\_inelastic} = 1099$ kip·in

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

<u>3 - 3 AXIS</u>

<u>2 - 2 AXIS</u>

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2}\right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 1089$ kip·in

$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$

$M_{n2} := M_{p2}$

$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$M_{n3} = 1089$ kip·in $\qquad$ $M_{n2} = 603$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right] \qquad \text{(eq. LRFD A-F1-3)}$$

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right] \qquad \text{(eq. LRFD A-F1-3)}$$

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 1089$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS	2 - 2 AXIS
$M_{lt3} := M_3$	$M_{lt2} := M_2$
$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$	$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$
$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^2}$	$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^2}$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_3 = 1$ $B2_2 = 1$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 266$ kip·in $M_{u2} = 150$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.555$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2 \cdot \frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.499$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u> <u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

Flanges will not buckle due to shear as in the web. Therefore:

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

$$V_{n2} := \text{if}\left(\lambda \le r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \le \lambda)\cdot(\lambda \le r2), 0.6 \cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$$\phi V_{n3} := \phi_v \cdot V_{n3} \qquad \phi V_{n2} := \phi_v \cdot V_{n2}$$

$$\phi V_{n3} = 156 \quad \text{kips} \qquad \phi V_{n2} = 136 \quad \text{kips}$$

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$A_o := \left(b_f - t_f\right)\cdot\left(d - t_w\right)$$

$$f_{v3} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_3}{2 \cdot A_f} \qquad f_{v2} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_2}{2 \cdot A_w}$$

$$f_{v3} = 1.61 \quad \text{ksi} \qquad f_{v2} = 1.99 \quad \text{ksi}$$

$$\frac{f_{v3}}{F_v} = 0.083 \qquad \frac{f_{v2}}{F_v} = 0.102$$

Bending / Axial DCR controls

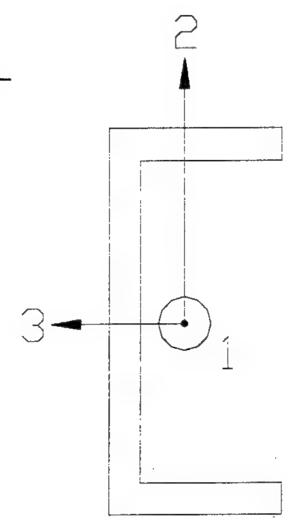
$$DCR := \begin{bmatrix} C1 \\ \dfrac{f_{v3}}{F_v} \\ \dfrac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 0.499$$

APPENDIX B14

Type L Channel at the Top of Cab Window (C6 x 8.2) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for C-shaped members
-Note member axis definitions below



the 1 axis is the longitudinal axis out of the page

TOWER : Type L, 50 ft. (Salinas, CA)

MEMBER : C6x8.2 ; channel at top of window
strong axis bending & axial force only

ANALYSIS RUN : L 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -12.5\ \text{kips}$ $\qquad M_1 := 1\ \text{kip}\cdot\text{in}$

$V_2 := -9.4\ \text{kips}$ $\qquad M_2 := 0\ \text{kip}\cdot\text{in}$

$V_3 := -5.2\ \text{kips}$ $\qquad M_3 := -387\ \text{kip}\cdot\text{in}$

$P_2 := |V_2|$

$M_1 := |M_1| \qquad M_2 := |M_2| \qquad M_3 := |M_3|$

$P_3 := |V_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 2.40\ in^2$	$d := 6.00$ in	$k_{nt3} := 1$	
$A_e := A$	$k := 0 + \frac{13}{16}$ in	$k_{nt2} := 1$	K values by LRFD Table C-C2.1
$I_3 := 13.1\ in^4$	$t_w := 0.20$ in	$k_{lt3} := 1.0$	
$I_2 := 0.693\ in^4$	$b_f := 1.920$ in	$k_{lt2} := 1.0$	
$S_3 := 4.38\ in^3$	$t_f := 0.343$ in	$k_1 := 1.0$	torsional effective length factor
$S_2 := 0.492\ in^3$	$x_o := 0.511$ in	$L_{b3} := 237$ in	
$Z_3 := 5.13\ in^3$	$e_o := 0.599$ in	$L_{b2} := 12$ in	
$Z_2 := 0.993\ in^3$		$L_{b1} := 12$ in	unbraced length for torsion

MATERIAL & CODE PROPERTIES

$E := 29000$ ksi	$Fy := 36$ ksi	$\phi_b := 0.90$	$\phi_c := 0.85$
$G := 11200$ ksi	$Fu := 58$ ksi	$\phi_v := 0.90$	$C_b := 1.0$
$Fr := 10$ ksi	$Fyf := Fy$	$\phi_{ty} := 0.90$	C_b is conservatively taken equal to 1.0
$F_L := Fyf - Fr$	$Fyw := Fy$	$\phi_{tf} := 0.75$	

flexural-torsional properties (pg 1-145)

$C_w := 4.72$	in^6	warping constant
$J := 0.08$	in^4	torsional constant
$H := 0.824$		flexural constant
$r_o := 2.65$	in	polar radius of gyration about shear center

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 2 \cdot k$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, \left(\phi_{tf}\cdot Fu\cdot A_e\right), \phi_{ty}\cdot Fy\cdot A\right]$$ (eq. LRFD D1-1) (eq. LRFD D1-2)

$\phi P_{nt} = 77.8$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy-10}}$$

$\lambda_{p\_flange} = 10.8$ $\qquad \lambda_{r\_flange} = 27.7$

$\lambda_{flange} := \dfrac{b_f}{t_f}$ $\qquad \lambda_{flange} = 5.6$ $\qquad \lambda_{flange} < \lambda_{p\_flange}$ OK

Webs in combined flexure and compression (for h/t ratio)

$P_u := |P_1|$

$P_y := Fy\cdot A$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b\cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b\cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75\cdot P_u}{\phi_b\cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74\cdot\frac{P_u}{\phi_b\cdot P_y}\right)$$

$\lambda_{p\_web} = 69.1$ $\qquad \lambda_{r\_web} = 142$

member values

$\lambda_{web} := \dfrac{h}{t_w}$ $\qquad \lambda_{web} = 21.9$ $\qquad \lambda_{web} < \lambda_{p\_web}$ OK

$$\lambda_c := \text{if}\left(\frac{k_{lt3}\cdot L_{b3}}{r_3} > \frac{k_{lt2}\cdot L_{b2}}{r_2}, \frac{k_{lt3}\cdot L_{b3}}{r_3\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}, \frac{k_{lt2}\cdot L_{b2}}{r_2\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 1.14$

$$F_{crFB} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{crFB} = 20.9$ ksi

CHECK FLEXURAL-TORSIONAL BUCKLING

$$F_{ey} := \frac{\pi^2 \cdot E}{\left(\frac{k_{lt3} \cdot L_{b3}}{r_3}\right)^2}$$ y represents axis of symmetry

$$F_{ez} := \frac{\left[\frac{\pi^2 \cdot E \cdot C_w}{\left(k_1 \cdot L_{b1}\right)^2} + G \cdot J\right]}{A \cdot r_o^2}$$

$$F_e := \frac{F_{ey} + F_{ez}}{2 \cdot H} \cdot \sqrt{1 - \frac{4 \cdot F_{ey} \cdot F_{ez} \cdot H}{\left(F_{ey} + F_{ez}\right)^2}}$$

$$\lambda_e := \sqrt{\frac{Fy}{F_e}}$$

$$F_{crTB} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$

$$F_{cr} := \text{if}\left(F_{crTB} < F_{crFB}, F_{crTB}, F_{crFB}\right)$$

$$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$$ (eq.LRFD E2-1)

$$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$$

$\phi P_n = 42.7$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 185$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 26.6$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L{}^2}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS

2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}}\right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 185$ kip·in

$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$

$M_{n2} := M_{p2}$

$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$

$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$

$M_{n3} = 185$ kip·in

$M_{n2} = 26.6$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right]$$ (eq. LRFD A-F1-3)

$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$

$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$

$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$

$M_{n3} = 185$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$$B2_3 = 1.23$$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$$B2_2 = 1.01$$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 476$ kip·in $\qquad$ $M_{u2} = 0$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) $\qquad$ (eq. LRFD H1-1b)

$C = 2.84$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 2.54$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u>

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf\cdot A_f$$

<u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \text{if}\left(\lambda \leq r1, 0.6\cdot Fyw\cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \leq \lambda, \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 28.4$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3}$

$\phi V_{n3} = 25.6$ kips

$V_{n2} = 25.9$ kips

$\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n2} = 23.3$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1 + P_2 \cdot (e_o + x_o)}{A_f \cdot \frac{d}{2}} + \frac{P_3}{A_f} \qquad f_{v2} := \frac{P_2}{A_w}$$

$$f_{v3} = 6.84 \ \text{ksi} \qquad f_{v2} = 7.83 \ \text{ksi}$$

$$\frac{f_{v3}}{F_v} = 0.352 \qquad \frac{f_{v2}}{F_v} = 0.403$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 2.54$$

APPENDIX B15

Type L Upgrade Structural Tubing (TS 20 x 4 x 1/2) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

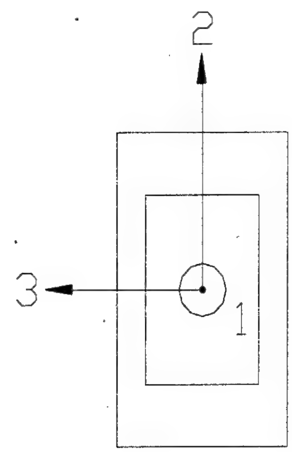
NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Type L, 30 ft. (San Carlos, CA)

MEMBER : Corner Mullion @ roof

ANALYSIS RUN : L 12

APPLIED LOADS : 100% NEHRP-97 + self weight



the 1 axis is the longitudinal axis out of the page

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -14.6$ kips $\quad M_1 := 125$ kip·in

$V_2 := 101.7$ kips $\quad M_2 := 68$ kip·in

$V_3 := 2.8$ kips $\quad M_3 := 2423$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3| \quad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$d := 20$ in	$k_{nt3} := 1.0$		
$t_w := 0.5$ in	$k_{nt2} := 1.0$	K values by LRFD Table C-C2.1	
$b_f := 4.0$ in	$k_{lt3} := 1.0$		
$t_f := 0.5$ in	$k_{lt2} := 1.0$		
$J := 205.0$ in^4	$L_{b3} := 24.0$ in		
	$L_{b2} := 24.0$ in		

MATERIAL & CODE PROPERTIES

$E := 29000$ ksi	$Fy := 36$ ksi	$\phi_b := 0.90$	$\phi_c := 0.85$
$G := 11200$ ksi	$Fu := 58$ ksi	$\phi_v := 0.90$	$C_b := 1.0$
$Fr := 10$ ksi	$Fyf := Fy$	$\phi_{ty} := 0.90$	This value of Cb=1 is conservatively taken
$F_L := Fyf - Fr$	$Fyw := Fy$	$\phi_{tf} := 0.75$	

CALCULATED PROPERTIES

$A := 22.4$ in^2	$S_3 := 88.9$ in^4	$A_w := 2 \cdot d \cdot t_w$
$A_e := A$	$S_2 := 30.8$ in^4	$A_f := 2 \cdot b_f \cdot t_f$
$I_3 := 889$ in^3	$Z_3 := 123$ in^4	$h := d - 3 \cdot t_f$
$I_2 := 61.6$ in^3	$Z_2 := 36.0$ in^4	$b := b_f - 3 \cdot t_w$
$r_3 := 6.31$ in		
$r_2 := 1.66$ in		

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)

(eq. LRFD D1-2)

$\phi P_{nt} = 726$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$$

$$\lambda_{p\_flange} = 31.7 \qquad \lambda_{r\_flange} = 39.7$$

$$\lambda_{flange} := \frac{b}{t_f} \qquad \lambda_{flange} = 5 \qquad \lambda_{flange} < \lambda_{p\_flange} \text{ OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 101 \qquad \lambda_{r\_web} = 159$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 37 \qquad \lambda_{web} < \lambda_{p\_web} \text{ OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi}\cdot\sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi}\cdot\sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$$\lambda_c = 0.162$$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$$F_{cr} = 35.6 \quad \text{ksi}$$

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \mathrm{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$ $\phi P_n = 678$ kips

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BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \mathrm{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 4428$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \mathrm{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 1296$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-5)

$M_r := Fyf \cdot S_3$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-10)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right)\left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

$M_{nLTB\_inelastic} = 4474$ kip·in

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS | 2 - 2 AXIS

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2}\right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := \mathrm{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 4428$ kip·in

$$M_{n3} := \mathrm{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \mathrm{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} := \mathrm{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$M_{n3} = 4428$ kip·in

$M_{n2} = 1296$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \mathrm{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \mathrm{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \mathrm{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \mathrm{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 4428$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\;2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$$B2_3 = 1$$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\;2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$$B2_2 = 1$$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 2423$ kip·in

$M_{u2} = 68$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.677$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.609$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

Flanges will not buckle due to shear as in the web. Therefore:

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n3} := 0.6\cdot Fyf\cdot A_f$$

$$V_{n2} := \text{if}\left(\lambda \leq r1, 0.6\cdot Fyw\cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \leq \lambda, \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$$\phi V_{n3} := \phi_v\cdot V_{n3} \qquad \phi V_{n2} := \phi_v\cdot V_{n2}$$

$\phi V_{n3} = 78$ kips $\qquad \phi V_{n2} = 389$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6\cdot\phi_v\cdot Fy$$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$A_o := (b_f - t_f)\cdot(d - t_w)$$

$$f_{v3} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_3}{A_f}$$

$$f_{v3} = 2.53 \quad \text{ksi}$$

$$\frac{f_{v3}}{F_v} = 0.13$$

$$f_{v2} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_2}{A_w}$$

$$f_{v2} = 6.92 \quad \text{ksi}$$

$$\frac{f_{v2}}{F_v} = 0.356$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$$\max(DCR) = 0.609$$

APPENDIX C

Member Properties and Loads for San Luis Obispo ATCT

Member Name	Description	Member Length (in.)	Weight per Length (k/in.)	Mass Per Length (k-s2/in2)
Intermediate 2				
I214	W16X40	45.25	0.02740	0.00007091
I220	W16X40	45.25	0.02740	0.00007091
I221	W14X34	84.00	0.02740	0.00007091
I222	W14X34	6.50	0.02740	0.00007091
I223	W14X34	50.00	0.01770	0.00004580
I224	W14X34	90.50	0.01770	0.00004580
I219	W16X45	60.00	0.01770	0.00004580
I216	W16X45	6.00	0.01770	0.00004580
I213	W16X45	79.50	0.01770	0.00004580
I207	W16X45	85.50	0.00933	0.00002414
I204	W16X40	90.50	0.00933	0.00002414
I203	W16X40	50.00	0.00933	0.00002414
I205	W16X40	45.25	0.01903	0.00004925
I208	W16X40	22.63	0.01903	0.00004925
I210	W16X40	22.63	0.01903	0.00004925
I211	W16X40	50.00	0.01903	0.00004925
I201	W16X40	84.00	0.01903	0.00004925
I202	W16X40	6.50	0.01903	0.00004925
I206	W12X30	82.50	0.02572	0.00006658
I209	W12X30	115.50	0.02572	0.00006658
I212	W12X30	82.50	0.02572	0.00006658
I215	C12X20.7	90.50	0.02572	0.00006658
I217	W12X30	115.50	0.02572	0.00006658
I218	W12X30	66.00	0.02572	0.00006658
Intermediate 3				
I314	W16X40	45.25	0.02694	0.00006971
I319	W16X40	45.25	0.02694	0.00006971
I320	W14X34	84.00	0.02694	0.00006971
I321	W14X34	6.50	0.02694	0.00006971
I322	W14X34	50.00	0.01742	0.00004508
I323	W14X34	90.50	0.01742	0.00004508
I318	W16X45	66.00	0.01742	0.00004508
I313	W16X45	24.50	0.01742	0.00004508
I311	W16X45	50.00	0.01742	0.00004508
I307	W16X45	90.50	0.00921	0.00002383
I303	W16X40	50.00	0.00921	0.00002383
I304	W16X40	90.50	0.00921	0.00002383
I305	W16X40	45.25	0.01873	0.00004847
I309	W16X40	45.25	0.01873	0.00004847
I310	W16X40	50.00	0.01873	0.00004847
I301	W16X40	84.00	0.01873	0.00004847
I302	W16X40	6.50	0.01873	0.00004847
I306	W12X30	82.50	0.02524	0.00006532
I308	W12X30	115.50	0.02524	0.00006532
I312	W12X30	82.50	0.02524	0.00006532
I315	C12X20.7	90.50	0.02524	0.00006532
I316	W12X30	115.50	0.02524	0.00006532
I317	W12X30	66.00	0.02524	0.00006532
Junction				
J516	W16X40	11.00	0.03216	0.00008323
J519	W16X40	39.75	0.03216	0.00008323
J523	W16X40	39.75	0.03216	0.00008323
J524	W14X34	71.88	0.03216	0.00008323

J525	W14X34	12.13	0.03216	0.00008323
J526	W14X34	6.50	0.03216	0.00008323
J527	W14X34	50.00	0.02094	0.00005420
J528	W14X34	6.50	0.02094	0.00005420
J529	W14X34	84.00	0.02094	0.00005420
J522	W16X45	66.00	0.02094	0.00005420
J517	W16X45	24.50	0.02094	0.00005420
J514	W16X45	50.00	0.02094	0.00005420
J511	W16X45	45.25	0.01127	0.00002917
J507	W16X45	45.25	0.01127	0.00002917
J503	W16X40	50.00	0.01127	0.00002917
J504	W16X40	6.50	0.01127	0.00002917
J505	W16X40	84.00	0.01127	0.00002917
J501	W16X40	84.00	0.02249	0.00005820
J502	W16X40	6.50	0.02249	0.00005820
J506	W16X40	43.81	0.02249	0.00005820
J510	W16X40	43.81	0.02249	0.00005820
J512	W16X40	2.88	0.02249	0.00005820
J513	W16X40	50.00	0.02249	0.00005820
J508	W12X30	82.50	0.02974	0.00007696
J509	W12X30	115.50	0.02974	0.00007696
J515	W12X30	82.50	0.02974	0.00007696
J518	C12X20.7	90.50	0.02974	0.00007696
J520	W12X30	115.50	0.02974	0.00007696
J521	W12X30	66.00	0.02974	0.00007696
Cab Access				
CA701	W18X40	75.75	0.01415	0.00003662
CA702	W18X40	75.75	0.01415	0.00003662
CA703	W8X10	29.25	0.01415	0.00003662
CA704	W18X40	13.50	0.01415	0.00003662
CA705	W10X26	84.00	0.01415	0.00003662
CA706	W10X12	8.97	0.01415	0.00003662
CA707	W10X26	91.79	0.01415	0.00003662
CA708	W18X40	66.00	0.01415	0.00003662
CA709	W10X26	57.03	0.01415	0.00003662
CA710	W8X10	75.13	0.01415	0.00003662
CA711	W10X26	31.07	0.01415	0.00003662
CA712	W10X26	52.93	0.01415	0.00003662
Top of Shaft				
TS828	W16X40	73.94	0.03313	0.00008573
TS832	W16X77	71.88	0.03313	0.00008573
TS833	W16X77	43.63	0.03313	0.00008573
TS834	W16X77	31.50	0.02508	0.00006490
TS835	W16X77	10.06	0.02508	0.00006490
TS836	W16X77	73.94	0.02508	0.00006490
TS829	W16X40	66.00	0.02508	0.00006490
TS824	W16X40	7.94	0.02508	0.00006490
TS810	W16X40	73.93	0.01667	0.00004313
TS814	W16X40	16.56	0.01667	0.00004313
TS816	W16X40	50.00	0.01667	0.00004313
TS819	W16X40	16.56	0.01667	0.00004313
TS804	W16X77	31.50	0.01667	0.00004313
TS805	W16X77	10.06	0.01667	0.00004313
TS806	W16X77	73.94	0.01667	0.00004313
TS801	W16X77	73.94	0.02472	0.00006397
TS802	W16X77	16.56	0.02472	0.00006397
TS803	W16X77	25.00	0.02472	0.00006397
TS809	W16X40	73.94	0.02472	0.00006397
TS813	W16X40	13.69	0.02472	0.00006397
TS817	W16X40	63.88	0.02472	0.00006397
TS823	W16X40	5.56	0.02472	0.00006397
TS807	W8X35	82.02	0.01025	0.00002654
TS808	W10X26	82.02	0.01025	0.00002654
TS811	W8X35	22.54	0.01025	0.00002654

TS812	W10X26	22.54	0.01025	0.00002654
TS815	W10X26	56.00	0.01025	0.00002654
TS818	W10X26	39.00	0.01025	0.00002654
TS820	W14X34	53.56	0.01025	0.00002654
TS821	W14X34	64.19	0.01025	0.00002654
TS822	W14X34	29.25	0.01025	0.00002654
TS825	W10X26	20.00	0.01025	0.00002654
TS826	W10X26	22.54	0.01025	0.00002654
TS827	W10X26	22.54	0.01025	0.00002654
TS830	W10X26	82.02	0.01025	0.00002654
TS831	W10X26	82.02	0.01025	0.00002654
Cab Floor				
CF916	W10X26	42.69	0.01477	0.00003822
CF920	W10X26	21.50	0.01477	0.00003822
CF923	W10X26	57.61	0.01477	0.00003822
CF926	W10X26	57.61	0.01477	0.00003822
CF907	W10X26	8.13	0.00772	0.00001998
CF912	W10X26	42.69	0.00772	0.00001998
CF902	W10X26	57.61	0.00772	0.00001998
CF905	W10X26	57.61	0.00772	0.00001998
CF904	W10X26	57.61	0.01715	0.00004439
CF901	W10X26	57.61	0.01715	0.00004439
CF913	W10X26	47.88	0.01715	0.00004439
CF906	W10X26	8.13	0.01715	0.00004439
CF925	W10X26	57.61	0.02420	0.00006263
CF922	W10X26	57.61	0.02420	0.00006263
CF921	W10X26	20.00	0.02420	0.00006263
CF903	W14X34	66.13	0.01062	0.00002749
CF908	W10X26	37.63	0.01062	0.00002749
CF909	W10X26	61.94	0.01062	0.00002749
CF910	W10X26	49.78	0.01062	0.00002749
CF911	W10X26	49.78	0.01062	0.00002749
CF914	W10X26	85.38	0.01062	0.00002749
CF915	W14X34	85.38	0.01062	0.00002749
CF917	W10X26	61.94	0.01062	0.00002749
CF918	W8X40	49.78	0.01062	0.00002749
CF919	W8X40	49.78	0.01062	0.00002749
CF924	W14X34	79.50	0.01062	0.00002749
Bottom Window				
BW1003	TS7X7X3/16	57.61	0.00784	0.00002029
BW1001	TS7X7X3/16	57.61	0.00784	0.00002029
BW1002	TS7X7X3/16	57.61	0.00784	0.00002029
BW1004	TS7X7X3/16	57.61	0.00784	0.00002029
BW1006	TS7X7X3/16	57.50	0.00784	0.00002029
BW1008	TS7X7X3/16	57.50	0.00784	0.00002029
BW1010	TS7X7X3/16	57.61	0.00784	0.00002029
BW1012	TS7X7X3/16	57.61	0.00784	0.00002029
BW1011	TS7X7X3/16	57.61	0.00784	0.00002029
BW1009	TS7X7X3/16	57.61	0.00784	0.00002029
BW1007	TS7X7X3/16	57.50	0.00784	0.00002029
BW1005	TS7X7X3/16	57.50	0.00784	0.00002029
Top Window				
TW1103	TS7X7X3/16	72.35	0.00333	0.00000861
TW1101	TS7X7X3/16	72.35	0.00333	0.00000861
TW1102	TS7X7X3/16	72.35	0.00333	0.00000861
TW1104	TS7X7X3/16	72.35	0.00333	0.00000861
TW1106	TS7X7X3/16	72.38	0.00333	0.00000861
TW1108	TS7X7X3/16	72.38	0.00333	0.00000861
TW1110	TS7X7X3/16	72.35	0.00333	0.00000861
TW1112	TS7X7X3/16	72.35	0.00333	0.00000861
TW1111	TS7X7X3/16	72.35	0.00333	0.00000861
TW1109	TS7X7X3/16	72.35	0.00333	0.00000861
TW1107	TS7X7X3/16	72.38	0.00333	0.00000861
TW1105	TS7X7X3/16	72.38	0.00333	0.00000861

Cab Roof				
CR1232	W10X12	72.35	0.00597	0.00001545
CR1235	W10X12	72.35	0.00597	0.00001545
CR1223	W10X12	36.19	0.00597	0.00001545
CR1226	W10X12	36.19	0.00597	0.00001545
CR1210	W10X12	36.19	0.00363	0.00000940
CR1213	W10X12	36.19	0.00363	0.00000940
CR1202	W10X12	72.35	0.00363	0.00000940
CR1205	W10X12	72.35	0.00363	0.00000940
CR1204	W10X12	72.35	0.00649	0.00001679
CR1201	W10X12	72.35	0.00649	0.00001679
CR1211	W10X12	54.50	0.00649	0.00001679
CR1234	W10X12	72.35	0.00882	0.00002284
CR1231	W10X12	72.35	0.00882	0.00002284
CR1224	W10X12	54.50	0.00882	0.00002284
CR1217	W10X12	34.75	0.00882	0.00002284
CR1203	W14X30	72.38	0.00336	0.00000870
CR1206	W10X12	62.66	0.00336	0.00000870
CR1207	W10X12	62.66	0.00336	0.00000870
CR1208	W10X12	62.66	0.00336	0.00000870
CR1209	W10X12	62.66	0.00336	0.00000870
CR1212	W14X30	54.50	0.00336	0.00000870
CR1214	W10X12	62.66	0.00336	0.00000870
CR1215	W10X12	62.66	0.00336	0.00000870
CR1216	W14X30	17.88	0.00336	0.00000870
CR1218	W10X12	62.66	0.00336	0.00000870
CR1219	W10X12	62.66	0.00336	0.00000870
CR1220	W14X30	17.88	0.00336	0.00000870
CR1221	W10X12	62.66	0.00336	0.00000870
CR1222	W10X12	62.66	0.00336	0.00000870
CR1225	W14X30	54.50	0.00336	0.00000870
CR1227	W10X12	62.66	0.00336	0.00000870
CR1228	W10X12	62.66	0.00336	0.00000870
CR1229	W10X12	62.66	0.00336	0.00000870
CR1230	W10X12	62.66	0.00336	0.00000870
CR1233	W14X30	72.38	0.00336	0.00000870
Parapet				
PPT1303	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1301	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1302	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1304	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1306	TS7X7X3/16	72.38	0.00075	0.00000194
PPT1308	TS7X7X3/16	72.38	0.00075	0.00000194
PPT1310	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1312	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1311	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1309	TS7X7X3/16	72.35	0.00075	0.00000194
PPT1307	TS7X7X3/16	72.38	0.00075	0.00000194
PPT1305	TS7X7X3/16	72.38	0.00075	0.00000194

Intermediate Level 2

Summary:		
Eccentrically Distributed Loads:		
Weight from additional members =	374	lb.
Concrete Deck =	11567	lb.
Interior Walls =	8200	lb. (1/3 floor below + 2/3 floor above)
Stairs =	4282	lb.
Mech & Elect =	1334	lb. (5 psf over floor area)
Misc =	2669	lb. (10 psf over floor area)
Sub-total :	28426	lb.
Distribution to exterior beams:	14213	lb.
Distribution to interior beams:	14213	lb.
Total length of exterior beams:	77.00	ft.

Total length of interior beams:	46.04	ft.
Uniformly Distributed Loads:		
Wt. of exterior walls:	2755	lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)
section A-C = 1.5874 section C-E = 0.9566
section E-G = 0.4126 section G-A = 1.0434

Intermediate Level 2 Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Section
I214	W16X40	45.25	328.78	0.027398	
I220	W16X40	45.25	328.78	0.027398	A-C
I221	W14X34	84.00	328.78	0.027398	
I222	W14X34	6.50	328.78	0.027398	
I223	W14X34	50.00	212.35	0.017695	
I224	W14X34	90.50	212.35	0.017695	
I219	W16X45	60.00	212.35	0.017695	C-E
I216	W16X45	6.00	212.35	0.017695	
I213	W16X45	79.50	212.35	0.017695	
I207	W16X45	85.50	111.93	0.009328	
I204	W16X40	90.50	111.93	0.009328	E-G
I203	W16X40	50.00	111.93	0.009328	
I205	W16X40	45.25	228.37	0.019031	
I208	W16X40	22.63	228.37	0.019031	
I210	W16X40	22.63	228.37	0.019031	G-A
I211	W16X40	50.00	228.37	0.019031	
I201	W16X40	84.00	228.37	0.019031	
I202	W16X40	6.50	228.37	0.019031	

Intermediate Level 2 Interior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
I206	W12X30	82.50	308.70	0.025725
I209	W12X30	115.50	308.70	0.025725
I212	W12X30	82.50	308.70	0.025725
I215	C12X20.7	90.50	308.70	0.025725
I217	W12X30	115.50	308.70	0.025725
I218	W12X30	66.00	308.70	0.025725

Total self weight of beams: 4371.86 lb.

Intermediate Level 3

Summary:		
Eccentrically Distributed Loads:		
Weight from additional members =	374	lb.
Concrete Deck =	11567	lb.
Interior Walls =	7663	lb. (1/3 floor below + 2/3 floor above)
Stairs =	4282	lb.
Mech & Elect =	1334	lb. (5 psf over floor area)
Misc =	2669	lb. (10 psf over floor area)
Sub-total :	27889	lb.
Distribution to exterior beams:	13944	lb.
Distribution to interior beams:	13944	lb.
Total length of exterior beams:	77.00	ft.
Total length of interior beams:	46.04	ft.
Uniformly Distributed Loads:		
Wt. of exterior walls:	2755	lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)
section A-C = 1.5874 section C-E = 0.9566
section E-G = 0.4126 section G-A = 1.0434

Intermediate Level 3 Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Section
I314	W16X40	45.25	323.24	0.026937	
I319	W16X40	45.25	323.24	0.026937	A-C
I320	W14X34	84.00	323.24	0.026937	
I321	W14X34	6.50	323.24	0.026937	
I322	W14X34	50.00	209.01	0.017417	
I323	W14X34	90.50	209.01	0.017417	
I318	W16X45	66.00	209.01	0.017417	C-E
I313	W16X45	24.50	209.01	0.017417	
I311	W16X45	50.00	209.01	0.017417	
I307	W16X45	90.50	110.49	0.009208	
I303	W16X40	50.00	110.49	0.009208	E-G
I304	W16X40	90.50	110.49	0.009208	
I305	W16X40	45.25	224.73	0.018727	
I309	W16X40	45.25	224.73	0.018727	
I310	W16X40	50.00	224.73	0.018727	G-A
I301	W16X40	84.00	224.73	0.018727	
I302	W16X40	6.50	224.73	0.018727	

Intermediate Level 3 Interior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
I306	W12X30	82.50	302.86	0.025239
I308	W12X30	115.50	302.86	0.025239
I312	W12X30	82.50	302.86	0.025239
I315	C12X20.7	90.50	302.86	0.025239
I316	W12X30	115.50	302.86	0.025239
I317	W12X30	66.00	302.86	0.025239

Total self weight of beams: 4372 lb.

Junction Level

Summary:

Eccentrically Distributed Loads:

Weight from additional members =	122	lb.
Concrete Deck =	11567	lb.
Interior Walls =	11621	lb. (1/3 floor below + 2/3 floor above)
Stairs =	4467	lb.
Mech & Elect =	1334	lb. (5 psf over floor area)
Misc =	3748	lb. (10 psf over floor area)
Sub-total :	32859	lb.
Distribution to exterior beams:	16430	lb.
Distribution to interior beams:	16430	lb.
Total length of exterior beams:	77.00	ft.
Total length of interior beams:	46.04	ft.

Uniformly Distributed Loads:

Wt. of exterior walls: 3636 lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 1.5874 section C-E = 0.9566

section E-G = 0.4126 section G-A = 1.0434

Junction Level Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Section
J516	W16X40	11.00	385.93	0.032161	
J519	W16X40	39.75	385.93	0.032161	
J523	W16X40	39.75	385.93	0.032161	A-C
J524	W14X34	71.88	385.93	0.032161	
J525	W14X34	12.13	385.93	0.032161	

J526	W14X34	6.50	385.93	0.032161	
J527	W14X34	50.00	251.33	0.020945	
J528	W14X34	6.50	251.33	0.020945	
J529	W14X34	84.00	251.33	0.020945	C-E
J522	W16X45	66.00	251.33	0.020945	
J517	W16X45	24.50	251.33	0.020945	
J514	W16X45	50.00	251.33	0.020945	
J511	W16X45	45.25	135.26	0.011272	
J507	W16X45	45.25	135.26	0.011272	
J503	W16X40	50.00	135.26	0.011272	E-G
J504	W16X40	6.50	135.26	0.011272	
J505	W16X40	84.00	135.26	0.011272	
J501	W16X40	84.00	269.86	0.022488	
J502	W16X40	6.50	269.86	0.022488	
J506	W16X40	43.81	269.86	0.022488	G-A
J510	W16X40	43.81	269.86	0.022488	
J512	W16X40	2.88	269.86	0.022488	
J513	W16X40	50.00	269.86	0.022488	

Junction Level Interior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
J508	W12X30	82.50	356.84	0.029737
J509	W12X30	115.50	356.84	0.029737
J515	W12X30	82.50	356.84	0.029737
J518	C12X20.7	90.50	356.84	0.029737
J520	W12X30	115.50	356.84	0.029737
J521	W12X30	66.00	356.84	0.029737

Total self weight of beams: 4372 lb.

Cab Access Level

Summary:

Eccentrically Distributed Loads:

Weight from additional members =	195	lb.
Concrete Deck =	2918	lb.
Interior Walls =	2890	lb. (1/3 floor below + 2/3 floor above)
Stairs =	2541	lb.
Mech & Elect =	496	lb. (5 psf over floor area)
Misc =	318	lb. (10 psf over grating)
Sub-total :	9356	lb.
Distribution to exterior beams:	0	lb.
Distribution to interior beams:	9356	lb. (treat all as interior beams)
Total length of exterior beams:	0.00	ft.
Total length of interior beams:	55.10	ft.

Uniformly Distributed Loads:

Wt. of exterior walls: 0 lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 1.0000 section C-E = 1.0000

section E-G = 1.0000 section G-A = 1.0000

Cab Access Level Interior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
CA701	W18X40	75.75	169.82	0.014151
CA702	W18X40	75.75	169.82	0.014151
CA703	W8X10	29.25	169.82	0.014151
CA704	W18X40	13.50	169.82	0.014151
CA705	W10X26	84.00	169.82	0.014151
CA706	W10X12	8.97	169.82	0.014151
CA707	W10X26	91.79	169.82	0.014151
CA708	W18X40	66.00	169.82	0.014151

CA709	W10X26	57.03	169.82	0.014151
CA710	W10X26	75.13	169.82	0.014151
CA711	W10X26	31.07	169.82	0.014151
CA712	W10X26	52.93	169.82	0.014151

Total self weight of beams: 1552 lb.

Top of Shaft

Summary:

Eccentrically Distributed Loads:

Weight from additional members =	18809	lb. (Walkway included.)
Concrete Deck =	0	lb.
Interior Walls =	6657	lb. (1/3 floor below + 2/3 floor above)
Stairs =	1103	lb.
Mech & Elect =	1334	lb. (5 psf over floor area)
Misc =	0	lb.
Sub-total :	27903	lb.

Distribution to exterior beams:	20927	lb. (Catwalk included.)
Distribution to interior beams:	6976	lb.
Total length of exterior beams:	77.00	ft.
Total length of interior beams:	56.69	ft.

Uniformly Distributed Loads:

Wt. of exterior walls: 2077 lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 1.3634 section C-E = 1.0079

section E-G = 0.6366 section G-A = 0.9921

Top of Shaft Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Section
TS828	W16X40	73.94	397.53	0.033127	
TS832	W16X77	71.88	397.53	0.033127	A-C
TS833	W16X77	43.63	397.53	0.033127	
TS834	W16X77	31.50	300.91	0.025075	
TS835	W16X77	10.06	300.91	0.025075	
TS836	W16X77	73.94	300.91	0.025075	C-E
TS829	W16X40	66.00	300.91	0.025075	
TS824	W16X40	7.94	300.91	0.025075	
TS810	W16X40	73.93	199.99	0.016666	
TS814	W16X40	16.56	199.99	0.016666	
TS816	W16X40	50.00	199.99	0.016666	E-G
TS819	W16X40	16.56	199.99	0.016666	
TS804	W16X77	31.50	199.99	0.016666	
TS805	W16X77	10.06	199.99	0.016666	
TS806	W16X77	73.94	199.99	0.016666	
TS801	W16X77	73.94	296.61	0.024718	
TS802	W16X77	16.56	296.61	0.024718	
TS803	W16X77	25.00	296.61	0.024718	
TS809	W16X40	73.94	296.61	0.024718	G-A
TS813	W16X40	13.69	296.61	0.024718	
TS817	W16X40	63.88	296.61	0.024718	
TS823	W16X40	5.56	296.61	0.024718	

Top of Shaft Interior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
TS807	W8X35	82.02	123.06	0.010255
TS808	W10X26	82.02	123.06	0.010255
TS811	W8X35	22.54	123.06	0.010255
TS812	W10X26	22.54	123.06	0.010255

TS815	W10X26	56.00	123.06	0.010255
TS818	W10X26	39.00	123.06	0.010255
TS820	W14X34	53.56	123.06	0.010255
TS821	W14X34	64.19	123.06	0.010255
TS822	W14X34	29.25	123.06	0.010255
TS825	W10X26	20.00	123.06	0.010255
TS826	W10X26	22.54	123.06	0.010255
TS827	W10X26	22.54	123.06	0.010255
TS830	W10X26	82.02	123.06	0.010255
TS831	W10X26	82.02	123.06	0.010255

Total self weight of beams: 6155 lb.

Cab Floor

Summary:

Eccentrically Distributed Loads:

Weight from additional members =	597	lb.
Concrete Deck =	10339	lb.
Interior Walls =	1062	lb. (1/3 floor below + 2/3 floor above)
Stairs =	193	lb.
Mech & Elect =	1193	lb. (5 psf over floor area)
Misc =	1000	lb. (Approximate weight of consoles based on Type L calculations.)
Sub-total :	14384	lb.

Distribution to exterior beams:	7192	lb.
Distribution to interior beams:	7192	lb.
Total length of exterior beams:	43.06	ft.
Total length of interior beams:	56.42	ft.

Uniformly Distributed Loads:

Wt. of exterior walls: 1054 lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 0.9143 section C-E = 0.4080

section E-G = 1.0857 section G-A = 1.5920

Cab Floor Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Section
CF916	W10X26	42.69	177.20	0.014767	
CF920	W10X26	21.50	177.20	0.014767	A-C
CF923	W10X26	57.61	177.20	0.014767	
CF926	W10X26	57.61	177.20	0.014767	
CF907	W10X26	8.13	92.63	0.007719	
CF912	W10X26	42.69	92.63	0.007719	C-E
CF902	W10X26	57.61	92.63	0.007719	
CF905	W10X26	57.61	92.63	0.007719	
CF904	W10X26	57.61	205.83	0.017153	
CF901	W10X26	57.61	205.83	0.017153	
CF913	W10X26	47.88	205.83	0.017153	E-G
CF906	W10X26	8.13	205.83	0.017153	
CF925	W10X26	57.61	290.40	0.024200	
CF922	W10X26	57.61	290.40	0.024200	G-A
CF921	W10X26	20.00	290.40	0.024200	

Cab Floor Interior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
CF903	W14X34	66.13	127.48	0.010623
CF908	W10X26	37.63	127.48	0.010623
CF909	W10X26	61.94	127.48	0.010623
CF910	W10X26	49.78	127.48	0.010623
CF911	W10X26	49.78	127.48	0.010623
CF914	W10X26	85.38	127.48	0.010623
CF915	W14X34	85.38	127.48	0.010623

CF917	W10X26	61.94	127.48	0.010623
CF918	W8X40	49.78	127.48	0.010623
CF919	W8X40	49.78	127.48	0.010623
CF924	W14X34	79.50	127.48	0.010623

Total self weight of beams: 3149 lb.

Bottom of Window

Summary:

Eccentrically Distributed Loads:

Exterior Walls =	5418	lb. (1/3 floor below + 2/3 floor above)
Misc =	0	lb.
Sub-total :	5418	lb.

Total length of exterior beams: 57.58 ft.

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 1.0000 section C-E = 1.0000

section E-G = 1.0000 section G-A = 1.0000

Bottom of Window Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
BW1003	TS7X7X3/16	57.61	94.10	0.007841
BW1001	TS7X7X3/16	57.61	94.10	0.007841
BW1002	TS7X7X3/16	57.61	94.10	0.007841
BW1004	TS7X7X3/16	57.61	94.10	0.007841
BW1006	TS7X7X3/16	57.50	94.10	0.007841
BW1008	TS7X7X3/16	57.50	94.10	0.007841
BW1010	TS7X7X3/16	57.61	94.10	0.007841
BW1012	TS7X7X3/16	57.61	94.10	0.007841
BW1011	TS7X7X3/16	57.61	94.10	0.007841
BW1009	TS7X7X3/16	57.61	94.10	0.007841
BW1007	TS7X7X3/16	57.50	94.10	0.007841
BW1005	TS7X7X3/16	57.50	94.10	0.007841

Total self weight of beams: 983 lb.

Top of Window

Summary:

Eccentrically Distributed Loads:

Exterior Walls =	2890	lb. (1/3 floor below + 2/3 floor above)
Misc =	0	lb.
Sub-total :	2890	lb.

Total length of exterior beams: 72.36 ft.

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 1.0000 section C-E = 1.0000

section E-G = 1.0000 section G-A = 1.0000

Top of Window Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
TW1103	TS7X7X3/16	72.35	39.94	0.003328
TW1101	TS7X7X3/16	72.35	39.94	0.003328
TW1102	TS7X7X3/16	72.35	39.94	0.003328
TW1104	TS7X7X3/16	72.35	39.94	0.003328
TW1106	TS7X7X3/16	72.38	39.94	0.003328
TW1108	TS7X7X3/16	72.38	39.94	0.003328
TW1110	TS7X7X3/16	72.35	39.94	0.003328
TW1112	TS7X7X3/16	72.35	39.94	0.003328
TW1111	TS7X7X3/16	72.35	39.94	0.003328
TW1109	TS7X7X3/16	72.35	39.94	0.003328
TW1107	TS7X7X3/16	72.38	39.94	0.003328
TW1105	TS7X7X3/16	72.38	39.94	0.003328

Total self weight of beams: 1236 lb.

Cab Roof

Summary:

Eccentrically Distributed Loads:

Weight from additional members =	263	lb.
Roof Deck =	5687	lb.
Interior Walls =	0	lb. (1/3 floor below + 2/3 floor above)
Stairs =	0	lb.
Mech & Elect =	1889	lb. (5 psf over floor area)
Misc =	0	lb.
Sub-total :	7840	lb.
Distribution to exterior beams:	3920	lb.
Distribution to interior beams:	3920	lb.
Total length of exterior beams:	72.28	ft.
Total length of interior beams:	97.22	ft.

Uniformly Distributed Loads:

Wt. of exterior walls:	1482	lb. (1/3 floor below + 2/3 floor above)

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 0.9426 section C-E = 0.4256

section E-G = 1.0574 section G-A = 1.5744

Cab Roof Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)	Section
CR1232	W10X12	72.35	71.63	0.005969	
CR1235	W10X12	72.35	71.63	0.005969	A-C
CR1223	W10X12	36.19	71.63	0.005969	
CR1226	W10X12	36.19	71.63	0.005969	
CR1210	W10X12	36.19	43.59	0.003632	
CR1213	W10X12	36.19	43.59	0.003632	C-E
CR1202	W10X12	72.35	43.59	0.003632	
CR1205	W10X12	72.35	43.59	0.003632	
CR1204	W10X12	72.35	77.85	0.006488	
CR1201	W10X12	72.35	77.85	0.006488	E-G
CR1211	W10X12	54.50	77.85	0.006488	
CR1234	W10X12	72.35	105.89	0.008824	
CR1231	W10X12	72.35	105.89	0.008824	
CR1224	W10X12	54.50	105.89	0.008824	G-A
CR1217	W10X12	34.75	105.89	0.008824	

Cab Roof Interior Beams: Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
CR1203	W14X30	72.38	40.32	0.003360
CR1206	W10X12	62.66	40.32	0.003360
CR1207	W10X12	62.66	40.32	0.003360
CR1208	W10X12	62.66	40.32	0.003360
CR1209	W10X12	62.66	40.32	0.003360
CR1212	W14X30	54.50	40.32	0.003360
CR1214	W10X12	62.66	40.32	0.003360
CR1215	W10X12	62.66	40.32	0.003360
CR1216	W14X30	17.88	40.32	0.003360
CR1218	W10X12	62.66	40.32	0.003360
CR1219	W10X12	62.66	40.32	0.003360
CR1220	W14X30	17.88	40.32	0.003360
CR1221	W10X12	62.66	40.32	0.003360
CR1222	W10X12	62.66	40.32	0.003360
CR1225	W14X30	54.50	40.32	0.003360
CR1227	W10X12	62.66	40.32	0.003360
CR1228	W10X12	62.66	40.32	0.003360
CR1229	W10X12	62.66	40.32	0.003360
CR1230	W10X12	62.66	40.32	0.003360
CR1233	W14X30	72.38	40.32	0.003360

Total self weight of beams:	2468	lb.	

Parapet

Summary:		
Eccentrically Distributed Loads:		
Exterior Walls =	652	lb. (1/3 floor below)
Misc =	0	lb.
Sub-total :	652	lb.

Total length of exterior beams: 72.36 ft.

Eccentricity Factors: (Refer to calculations for locations of sections)

section A-C = 1.0000 section C-E = 1.0000

section E-G = 1.0000 section G-A = 1.0000

Parapet Exterior Beams:

Member Name	Section	Length (in.)	Additional Load (lb./ft.)	Additional Load (k/in)
PPT1303	TS7X7X3/16	72.35	9.02	0.000751
PPT1301	TS7X7X3/16	72.35	9.02	0.000751
PPT1302	TS7X7X3/16	72.35	9.02	0.000751
PPT1304	TS7X7X3/16	72.35	9.02	0.000751
PPT1306	TS7X7X3/16	72.38	9.02	0.000751
PPT1308	TS7X7X3/16	72.38	9.02	0.000751
PPT1310	TS7X7X3/16	72.35	9.02	0.000751
PPT1312	TS7X7X3/16	72.35	9.02	0.000751
PPT1311	TS7X7X3/16	72.35	9.02	0.000751
PPT1309	TS7X7X3/16	72.35	9.02	0.000751
PPT1307	TS7X7X3/16	72.38	9.02	0.000751
PPT1305	TS7X7X3/16	72.38	9.02	0.000751

Total self weight of beams: 1236 lb.

ADDITIONAL MEMBERS: (Refer to hand calculations)

Intermediate Level 2:(I2)	W10x26	374	lb.
Intermediate Level 3:(I3)	W10x26	374	lb.
Junction Level:(J)	W10x26	122	lb.
Cab Access Level:(CA)	W16x26	195	lb.
Top of Shaft:(TS)	walkway (next sect.)	18629	lb.
	W10x26	179	lb.
	Total:	18809	lb.
Cab Floor:(CF)	W10x26	441	lb.
	W8x10	35	lb.
	L4x4x1/4	121	lb.
	Total:	597	lb.
Cab Roof:(CR)	W8x10	89	lb.
	W10x12	173	lb.
	Total:	263	lb.
Walkway:	(load to TS)		
	C12x20.7	1739	lb.
	MC8x18.7	1111	lb.
	C8x11.5	309	lb.
	L4x4x1/4	1075	lb.
	concrete deck	12507	lb.
	railing	1658	lb.
	kick plate	230	lb.
	Total:	18629	lb.

SLAB:

effective slab depth=	3.25	in. (Accounts for decking indents.)
steel deck=	2.72	lb./sq. ft. (Vulcraft p. 33)
floor slab unit weight=	43.35	lb./sq. ft.

ROOF:	steel deck=	2.72	lb./sq. ft. (Vulcraft p. 33)
	8" rigid insulation=	12.00	lb./sq. ft.
	single ply membrane=	0.33	lb./sq. ft. (LRFD, 7-7, 3 ply=1 lb/sq. ft.)
	total=	15.05	
GRATING:	unit weight:	11.08	lb./sq. ft. (Refer to hand calculations)

FLOOR AREAS: Take CL to CL distance of perimeter floor beams as the effective floor area.

	Area (sq. in.)	Area (sq. ft.)
Intermediate Level 2:	38428.50	266.86
Intermediate Level 2:	38428.50	266.86
Junction Level:	38428.50	266.86
Junction Grating:	---------	107.92
Cab access Grating:	---------	31.80
Cab access Slab:	9692.91	67.31
Top of Shaft:	38428.50	266.86
Cab Floor:	34349.06	238.54
Cab Roof:	54416.95	377.90

INTERIOR WALLS: NOTES: -These numbers are very approximate; quick estimates were made from the drawi
-Door weights were taken to equal the wall weight -- this is conservative
-Windows were neglected -- conservative

Wall type	Unit Weight (lb./sq. ft.)	(Refer to hand calculations)
A	13.14	(insulation = .5psf per inch thickness)
B1	8.14	
B2	5.64	
C	9.06	

Interior wall weight resting on floor:

Ground:	length wall A	30.88	ft.	
	length wall B1	0.00	ft.	
	length wall B2	82.00	ft.	
	length wall C	10.03	ft.	
	wall height	9.67	ft.	
	Total Weight:	9274	lb.	
Intermediate Level 2:	length wall A	20.34	ft.	
	length wall B1	0.00	ft.	
	length wall B2	77.00	ft.	
	length wall C	10.03	ft.	
	wall height	9.67	ft.	
	Total Weight:	7663	lb.	
Intermediate Level 3:	length wall A	20.34	ft.	
	length wall B1	0.00	ft.	
	length wall B2	77.00	ft.	
	length wall C	10.03	ft.	
	wall height	9.67	ft.	
	Total Weight:	7663	lb.	
Junction:	length wall A	20.34	ft.	
	length wall B1	18.13	ft.	
	length wall B2	83.50	ft.	
	length wall C	6.00	ft.	
	wall height	8.81	ft.	
	Total Weight:	13600	lb.	
Cab Access to Junction:	length wall A	0.00	ft.	(Refer to hand calculations.)
	length wall B1	0.00	ft.	

	length wall B2	0.00	ft.	
	length wall C	21.00	ft.	
	wall height	8.17	ft.	
	Total Weight:	5318	lb.	(grating included here)
Cab Access:	length wall A	3.61	ft.	(Refer to hand calculations.)
	length wall B1	18.60	ft.	
	length wall B2	19.80	ft.	
	length wall C	0.00	ft.	
	wall height	8.17	ft.	
	Total Weight:	2890	lb.	(grating included here)
Top of Shaft:	length wall A	0.00	ft.	(Refer to hand calculations.)
	length wall B1	13.60	ft.	
	length wall B2	0.00	ft.	
	length wall C	0.00	ft.	
	wall height	5.58	ft.	
	Total Weight:	3186	lb.	

STAIRS: (Refer to hand calculations)

steps:	stair area=	18.00	sq. in.
	linear weight=	14.38	lb./ft.
subtread:	thickness=	0.10	in.
	width=	14.00	in.
	area=	1.46	sq. in.
	linear weight=	4.27	lb./ft.
riser:	thickness=	0.10	in.
	width=	8.00	in.
	area=	0.84	sq. in.
	linear weight=	2.44	lb./ft.
abrasive insert:	linear weight=	2.00	lb./ft.
L1.5x1.25x3/16:	linear weight=	1.64	lb./ft.
landings:	unit weight=	43.35	lb./sq. ft.
Additional Members:			
MC12x10.6	length I2, I3, J=	35.50	ft.
	weight I2, I3, J=	376	lb.
MC12x20.7	length I2, I3, J=	7.01	ft.
	weight I2, I3, J=	145	lb.
W10x26	length I2, I3, J=	7.01	ft.
	weight I2, I3, J=	182	lb.
2L3x3x1/4	linear weight=	9.80	lb./ft.
	weight I2, I3, J=	392	lb.
Additional Angles:	weight I2, I3, J=	83	lb.
	weight CA=	63	lb.
	weight TS=	198	lb.
Railing	unit weight=	2.27	lb./ft. (ASD p.1-93)
	weight I2, I3, J=	248	lb.
	weight CA=	31	lb.
	weight TS=	17	lb.

TS12x2x3/16	unit weight=	17.08	lb./ft.
	weight CA=	456	lb.
	weight TS=	260	lb.

Weight per level:

	# of treads	Average tread length (in.)	Landing Area (sq. in.)
Intermediate Level 2:	16	38.75	3133.66
Intermediate Level 3:	16	38.75	3133.66
Junction Level:	15	42.13	3835.88
Cab Access:	10	35.76	0.00
Top of Shaft:	6	32.50	750.00
Cab Floor:	3	31.25	0.00

	Total Tread Weight (lb.)	Landing Weight (lb.)	Additional Weight (lb.)	Stair Wall Weight (lb.)	Total (lb.)
Intermediate Level 2:	1277	943	1426	635	4282
Intermediate Level 3:	1277	943	1426	635	4282
Junction Level:	1302	1155	1426	584	4467
Cab Access:	737	0	550	1254	2541
Top of Shaft:	402	226	475	0	1103
Cab Floor:	193	0	0	0	193

EXTERIOR WALLS: NOTE: -Window, louver, and first floor door openings were neglected

unit weights:	exterior wall	3.50	lb./sq. ft. (Refer to hand calculations.)
	cab glass	15.00	lb./sq. ft. (Refer to hand calculations.)

Exterior wall weight above floor:

ground:	total wall length	81.42	ft.
	wall height	9.67	ft.
	total weight	2755	lb.
Intermediate Level 2:	total wall length	81.42	ft.
	wall height	9.67	ft.
	total weight	2755	lb.
Intermediate Level 3:	total wall length	81.42	ft.
	wall height	9.67	ft.
	total weight	2755	lb.
Junction Level:	total wall length	81.42	ft.
	wall height	14.31	ft.
	total weight	4077	lb.
Cab Access:	total wall length	81.42	ft.
	wall height	0.00	ft.
	total weight	0	lb.
Top of Shaft:	total wall length	59.40	ft.
	wall height	2.59	ft.
	total weight	1077	lb.
Cab Floor:	total wall length	59.40	ft.
	wall height	1.92	ft.
	total weight	1043	lb.

Bottom of Windows:	total wall length	65.00	ft.
	wall height	7.80	ft.
	total weight	7605	lb.
Top of Windows:	total wall length	73.00	ft.
	wall height	2.08	ft.
	total weight	532	lb.
Cab Roof:	total wall length	73.00	ft.
	wall height	3.83	ft.
	total weight	1957	lb.

APPENDIX D1

San Luis Obispo Shaft Brace (L6 x 6 x 1/2) Evaluation

(based on AISC-LRFD, 2nd ed. 1994, pp 6-277)

NOTES : -This sheet is for Single Angle bracing members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : L 6x6x1/2 ; Bracing at the base in compression

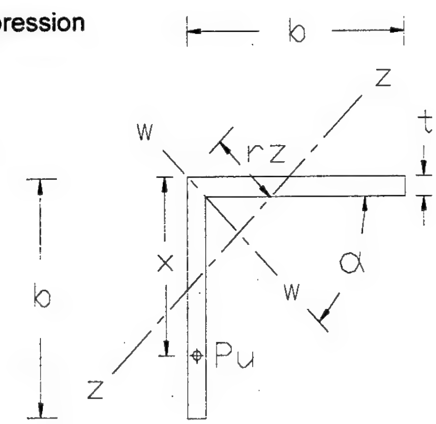
ANALYSIS RUN : SLO1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTION :

$P_u := 51.5$ kips

$P_u := |P_u|$



ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$b := 6.0$ in	$L := 12.24$ ft
$t := 0.5$ in	$x := 2.875$ in
$A_g := 5.75$ in^2	$y := 0.25$ in
$I_x := 19.9$ in^4	$\alpha := 45.0$ deg
$I_y := I_x$	
$r_z := 1.18$ in	

MATERIAL & CODE PROPERTIES

$F_y := 36 \text{ ksi}$ $\quad k := 1.0$ $\quad \phi := 0.9$

$E := 29000 \text{ ksi}$ $\quad C_b := 1.0$ $\quad \phi_b := 0.9$

$C_m := 1.0$

CALCULATED PROPERTIES

$\xi := x \cdot \sin(\alpha \cdot deg) - y \cdot \cos(\alpha \cdot deg)$ $\qquad \xi = 1.86$

$$\eta := x \cdot \cos(\alpha \cdot deg) - \left[y \cdot \frac{(\cos(\alpha \cdot deg))^2}{\sin(\alpha \cdot deg)} \right] + \frac{y}{\sin(\alpha \cdot deg)} \qquad \eta = 2.21$$

$w := \eta - r_z$ $\quad w = 1.03$ in $\quad$ distance from minor principal axis

$z := \xi$ $\quad z = 1.86$ in $\quad$ distance from major principal axis

$I_z := A_g \cdot r_z^2$

$I_w := I_x + I_y - I_z$

$r_w := \sqrt{\frac{I_w}{A_g}}$ $\qquad r_w = 2.35$

AXIAL CHECKS

COMPRESSION (AISC LRFD, p. 6-282)

check if slender element, determine reduction factor , Q

$$Q := \text{if}\left[\frac{b}{t} > 0.446 \cdot \sqrt{\frac{E}{F_y}}, \left(1.34 - 0.761 \cdot \frac{b}{t} \cdot \sqrt{\frac{F_y}{E}}\right), 1.0\right]$$

$$Q := \text{if}\left[\frac{b}{t} > 0.910 \cdot \sqrt{\frac{E}{F_y}}, 0.534 \cdot \frac{E}{F_y \cdot \left(\frac{b}{t}\right)^2}, Q\right] \qquad Q = 1$$

$$\lambda_c := \frac{k \cdot L \cdot 12}{r_z \cdot \pi} \cdot \sqrt{\frac{F_y}{E}} \qquad \lambda_c = 1.4$$

$$F_{cr} := \text{if}\left[\lambda_c \cdot \sqrt{Q} \le 1.5, Q \cdot \left[\left(0.658^{Q \cdot \lambda_c^{2}}\right) \cdot F_y\right], \frac{0.877}{\lambda_c^{2}} \cdot F_y\right] \qquad F_{cr} = 15.9 \quad \text{ksi}$$

$P_n := A_g \cdot F_{cr}$ nominal strength for compression

$P_n = 91.6$ kips

BENDING CHECK

Bending about principle axes (5.3, pp. 6-285)

$$M_{oB} := \frac{C_b \cdot 0.46 \cdot E \cdot b^2 \cdot t^2}{L \cdot 12.0} \qquad M_{oB} = 817 \quad \text{kip} \cdot \text{in} \qquad \text{(EQ. 5.5)}$$

$$C_w := b \cdot \sin(\alpha \cdot \text{deg})$$

$$M_y := F_y \cdot \frac{I_w}{C_w} \qquad M_y = 270 \quad \text{kip} \cdot \text{in}$$

5.1.3. Limit state of lateral-Torsional Buckling

$$M_{nwLTB} := \text{if}\left[M_{oB} \le M_y, \left(0.92 - 0.17 \cdot \frac{M_{oB}}{M_y}\right) \cdot M_{oB}, \left(1.58 - 0.83 \cdot \sqrt{\frac{M_y}{M_{oB}}}\right) \cdot M_y\right] \qquad \text{(EQ. 5.3a)} \ \text{(EQ. 5.3b)}$$

$$M_{nwLTB} := \text{if}\left(M_{nwLTB} > 1.25 \cdot M_y, 1.25 \cdot M_y, M_{nwLTB}\right)$$

$M_{nwLTB} = 298$ kip·in

5.1.1. Limit State of Local Buckling

$$S_w := \frac{I_w}{C_w}$$

$$M_{nwLB} := \text{if}\left[\frac{b}{t} > 0.382\cdot\sqrt{\frac{E}{F_y}}, F_y\cdot S_w\cdot\left[1.25 - 1.49\cdot\left[\frac{\frac{b}{t}}{0.382\cdot\sqrt{\frac{E}{F_y}}} - 1\right]\right], 1.25\cdot F_y\cdot S_w\right]$$

$$M_{nwLB} := \text{if}\left(\frac{b}{t} > 0.446\cdot\sqrt{\frac{E}{F_y}}, Q\cdot F_y\cdot S_w, M_{nwLB}\right)$$

$M_{nwLB} = 294$ kip·in

$$M_{nw} := \text{if}\left(M_{nwLB} < M_{nwLTB}, M_{nwLB}, M_{nwLTB}\right)$$

$M_{nw} = 294$ kip·in

Minor Principal Axis (z-z) Bending

$$C_{zTIP} := b\cdot\cos(\alpha\cdot deg) - \frac{t}{\sin(\alpha\cdot deg)}\cdot\left(\cos(\alpha\cdot deg)^2 - 1\right) - r_z$$

$C_{zTIP} = 3.42$

$$C_{zCORNER} := r_z$$

$$C_z := \text{if}\left(C_{zTIP} > C_{zCORNER}, C_{zTIP}, C_{zCORNER}\right)$$

$$S_z := \frac{I_z}{C_z}$$

$$M_{nzLB} := \text{if}\left[\frac{b}{t} > 0.382\cdot\sqrt{\frac{E}{F_y}}, F_y\cdot S_z\cdot\left[1.25 - 1.49\cdot\left[\frac{\frac{b}{t}}{0.382\cdot\sqrt{\frac{E}{F_y}}} - 1\right]\right], 1.25\cdot F_y\cdot S_z\right]$$

$$M_{nzLB} := \text{if}\left(\frac{b}{t} > 0.446\cdot\sqrt{\frac{E}{F_y}}, Q\cdot F_y\cdot S_z, M_{nzLB}\right)$$ $M_{nzLB} = 92$ kip·in

$$M_{nzy} := 1.25\cdot F_y\cdot S_z$$ $M_{nzy} = 105$ kip·in

$$M_{nz} := \text{if}\left(w > 0, M_{nzLB}, M_{nzy}\right)$$ $M_{nz} = 92$ kip·in

Determine Moment Amplification Factors

$$P_{e1w} := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot L \cdot 12.0}{r_w}\right)^2} \cdot A_g \qquad P_{e1w} = 422$$

$$B_{1w} := \frac{C_m}{1 - \frac{P_u}{P_{e1w}}} \qquad B_{1w} = 1.14$$

$$B_{1w} := \mathrm{if}\left(B_{1w} \geq 1.0, B_{1w}, 1.0\right)$$

$$M_{uw} := \left| B_{1w} \cdot z \cdot P_u \right| \qquad M_{uw} = 109 \quad \mathrm{kip \cdot in}$$

$$P_{e1z} := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot L \cdot 12.0}{r_z}\right)^2} \cdot A_g \qquad P_{e1z} = 106$$

$$B_{1z} := \frac{C_m}{1 - \frac{P_u}{P_{e1z}}} \qquad B_{1z} = 1.94$$

$$B_{1z} := \mathrm{if}\left(B_{1z} \geq 1.0, B_{1z}, 1.0\right)$$

$$M_{uz} := \left| B_{1z} \cdot w \cdot P_u \right| \qquad M_{uz} = 103 \quad \mathrm{kip \cdot in}$$

INTERACTION EQUATIONS:

$$DCR := \mathrm{if}\left[\frac{P_u}{\phi \cdot P_n} \geq 0.2, \frac{P_u}{\phi \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_{uw}}{\phi_b \cdot M_{nw}} + \frac{M_{uz}}{\phi_b \cdot M_{nz}}\right), \frac{P_u}{2 \cdot \phi \cdot P_n} + \left(\frac{M_{uw}}{\phi_b \cdot M_{nw}} + \frac{M_{uz}}{\phi_b \cdot M_{nz}}\right)\right]$$

$DCR = 2.1$ includes resistance factors

$$DCR := \mathrm{if}\left[\frac{P_u}{P_n} \geq 0.2, \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{uw}}{M_{nw}} + \frac{M_{uz}}{M_{nz}}\right), \frac{P_u}{2 \cdot P_n} + \left(\frac{M_{uw}}{M_{nw}} + \frac{M_{uz}}{M_{nz}}\right)\right]$$

$DCR = 1.89$ excludes resistance factors

APPENDIX D1a

San Luis Obispo Shaft Tension Only Brace (L6 x 6 x 1/2) Evaluation

(based on AISC-LRFD, 2nd ed. 1994, pp 6-277)

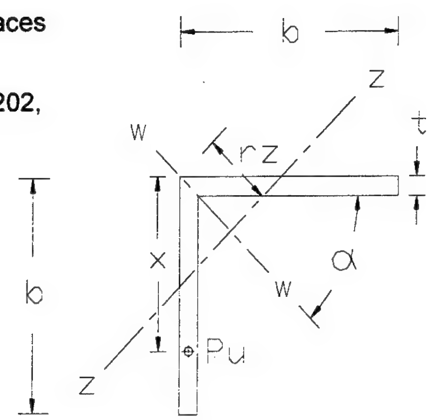
NOTES : -This sheet is for Single Angle bracing members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : L 6x6x1/2 ; Maximum tensile force in braces after buckling has occurred.

ANALYSIS RUN : Analysis of Critical Member BR202, case SLO3a

APPLIED LOADS : 100% NEHRP-97 + self weight



MAXIMUM REACTION :

$P_u := 109.4\,\text{kips}$

$P_u := |P_u|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$b := 6.0$ in

$t := 0.5$ in

$A_g := 5.75$ in^2

$I_x := 19.9$ in^4

$I_y := I_x$

$r_z := 1.18$ in

$L := 12.24$ ft

$x := 2.875$ in

$y := 0.25$ in

$\alpha := 45.0$ deg

MATERIAL & CODE PROPERITIES

$F_y := 36$ ksi

$E := 29000\,\text{ksi}$

$k := 1.0$

$C_b := 1.0$

$C_m := 1.0$

$\phi := 0.9$

$\phi_b := 0.9$

CALCULATED PROPERTIES

$\xi := x \cdot \sin(\alpha \cdot deg) - y \cdot \cos(\alpha \cdot deg)$ $\qquad \xi = 1.86$

$$\eta := x \cdot \cos(\alpha \cdot deg) - \left[y \cdot \frac{(\cos(\alpha \cdot deg))^2}{\sin(\alpha \cdot deg)} \right] + \frac{y}{\sin(\alpha \cdot deg)} \qquad \eta = 2.21$$

$w := \eta - r_z$ $\quad w = 1.03$ in $\quad$ distance from minor principal axis

$z := \xi$ $\quad z = 1.86$ in $\quad$ distance from major principal axis

$I_z := A_g \cdot r_z^2$

$I_w := I_x + I_y - I_z$

$$r_w := \sqrt{\frac{I_w}{A_g}} \qquad r_w = 2.35$$

AXIAL CHECKS

TENSION

$P_n := A_g \cdot F_y$

$P_n = 207$ kips

BENDING CHECK

Bending about principle axes (5.3, pp. 6-285)

$$M_{oB} := \frac{C_b \cdot 0.46 \cdot E \cdot b^2 \cdot t^2}{L \cdot 12.0} \qquad M_{oB} = 817 \qquad \text{kip} \cdot \text{in}$$

$$C_w := b \cdot \sin(\alpha \cdot deg)$$

$$M_y := F_y \cdot \frac{I_w}{C_w} \qquad M_y = 270 \qquad kip \cdot in$$

5.1.3. Limit state of lateral-Torsional Buckling

$$M_{nwLTB} := \text{if}\left[M_{oB} \le M_y, \left(0.92 - 0.17 \cdot \frac{M_{oB}}{M_y}\right) \cdot M_{oB}, \left(1.58 - 0.83 \cdot \sqrt{\frac{M_y}{M_{oB}}}\right) \cdot M_y\right]$$

$$M_{nwLTB} := \text{if}\left(M_{nwLTB} > 1.25 \cdot M_y, 1.25 \cdot M_y, M_{nwLTB}\right)$$

$$M_{nwLTB} = 298 \qquad kip \cdot in$$

5.1.1. Limit State of Local Buckling

check if slender element, determine reduction factor , Q

$$Q := \text{if}\left[\frac{b}{t} > 0.446 \cdot \sqrt{\frac{E}{F_y}}, \left(1.34 - 0.761 \cdot \frac{b}{t} \cdot \sqrt{\frac{E}{F_y}}\right), 1.0\right]$$

$$Q := \text{if}\left[\frac{b}{t} > 0.910 \cdot \sqrt{\frac{E}{F_y}}, 0.534 \cdot \frac{E}{F_y \cdot \left(\frac{b}{t}\right)^2}, Q\right] \qquad Q = 1$$

$$S_w := \frac{I_w}{C_w}$$

$$M_{nwLB} := \text{if}\left[\frac{b}{t} > 0.382 \cdot \sqrt{\frac{E}{F_y}}, F_y \cdot S_w \cdot \left[1.25 - 1.49 \cdot \left[\frac{\frac{b}{t}}{0.382 \cdot \sqrt{\frac{E}{F_y}}} - 1\right]\right], 1.25 \cdot F_y \cdot S_w\right]$$

$$M_{nwLB} := \text{if}\left(\frac{b}{t} > 0.446 \cdot \sqrt{\frac{E}{F_y}}, Q \cdot F_y \cdot S_w; M_{nwLB}\right)$$

$$M_{nwLB} = 294 \qquad kip \cdot in$$

$$M_{nw} := \text{if}\left(M_{nwLB} < M_{nwLTB}, M_{nwLB}, M_{nwLTB}\right)$$

$$M_{nw} = 294 \quad \text{kip} \cdot \text{in}$$

Minor Principal Axis (z-z) Bending

$$C_{zTIP} := b \cdot \cos(\alpha \cdot \text{deg}) - \frac{t}{\sin(\alpha \cdot \text{deg})} \cdot \left(\cos(\alpha \cdot \text{deg})^2 - 1\right) - r_z \qquad C_{zTIP} = 3.42$$

$$C_{zCORNER} := r_z$$

$$C_z := \text{if}\left(C_{zTIP} > C_{zCORNER}, C_{zTIP}, C_{zCORNER}\right)$$

$$S_z := \frac{I_z}{C_z}$$

$$M_{nzLB} := \text{if}\left[\frac{b}{t} > 0.382 \cdot \sqrt{\frac{E}{F_y}}, F_y \cdot S_z \cdot \left[1.25 - 1.49 \cdot \left[\frac{\frac{b}{t}}{0.382 \cdot \sqrt{\frac{E}{F_y}}} - 1\right]\right], 1.25 \cdot F_y \cdot S_z\right]$$

$$M_{nzLB} := \text{if}\left(\frac{b}{t} > 0.446 \cdot \sqrt{\frac{E}{F_y}}, Q \cdot F_y \cdot S_z, M_{nzLB}\right) \qquad M_{nzLB} = 92 \quad \text{kip} \cdot \text{in}$$

$$M_{nzy} := 1.25 \cdot F_y \cdot S_z \qquad M_{nzy} = 105 \quad \text{kip} \cdot \text{in}$$

$$M_{nz} := \text{if}\left(w > 0, M_{nzy}, M_{nzLB}\right) \qquad M_{nz} = 105 \quad \text{kip} \cdot \text{in}$$

Determine Moment Amplification Factors

$$B_{1w} := 1.0$$

$$M_{uw} := \left| B_{1w} \cdot z \cdot P_u \right| \qquad M_{uw} = 203 \quad \text{kip} \cdot \text{in}$$

$$B_{1z} := 1.0$$

$$B_{1z} := \text{if}\left(B_{1z} \geq 1.0, B_{1z}, 1.0\right)$$

$$M_{uz} := \left| B_{1z} \cdot w \cdot P_u \right| \qquad M_{uz} = 113 \quad \text{kip} \cdot \text{in}$$

INTERACTION EQUATIONS

$$DCR := \mathrm{if}\left[\frac{P_u}{\phi \cdot P_n} \ge 0.2, \frac{P_u}{\phi \cdot P_n} + \frac{8}{9}\cdot\left(\frac{M_{uw}}{\phi_b \cdot M_{nw}} + \frac{M_{uz}}{\phi_b \cdot M_{nz}}\right), \frac{P_u}{2\cdot\phi\cdot P_n} + \left(\frac{M_{uw}}{\phi_b \cdot M_{nw}} + \frac{M_{uz}}{\phi_b \cdot M_{nz}}\right)\right]$$

$DCR = 2.32$ includes resistance factors

$$DCR := \mathrm{if}\left[\frac{P_u}{P_n} \ge 0.2, \frac{P_u}{P_n} + \frac{8}{9}\cdot\left(\frac{M_{uw}}{M_{nw}} + \frac{M_{uz}}{M_{nz}}\right), \frac{P_u}{2\cdot P_n} + \left(\frac{M_{uw}}{M_{nw}} + \frac{M_{uz}}{M_{nz}}\right)\right]$$

$DCR = 2.09$ excludes resistance factors

Considering axial force alone:

$$DCR := \frac{P_u}{P_n}$$

$DCR = 0.529$ excluding bending due to eccentricity at connections

DISCUSSION

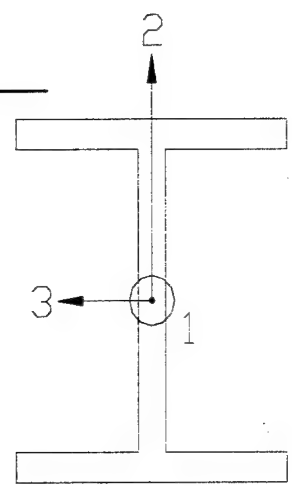
The L 6x6x1/2 is considered to be adequate even though the demand capacity ratio is slightly greater than 2.0 . The primary cause of the high DCR is the moment which is developed due to the eccentric connection. In the calculation of moment capacity about the major principal axis, local buckling controls. However, about the minor principal axis the limit state of yielding governs. In the calculation for the limit state of yielding, the AISC manual uses a shape factor of 1.25 . In the section of the AISC manual "Commentary on the Specification for Load and Resistance Factor Design of Single-Angle Members", the manual explains that the 1.25 shape factor used is less than the actual shape factor used in the plastic moment calculation for an angle. Therefore, exceeding 2.0 for this single tension member is considered to be acceptable.

APPENDIX D2

San Luis Obispo Shaft Column (W8 x 31) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for I-shaped members
-Note member axis definitions below



the 1 axis is the longitudinal axis out of the page

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : W8x31 ; Column at Base of Bent

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -175.3$ kips $\quad M_1 := 0$ kip·in

$V_2 := -0.8$ kips $\quad M_2 := -16$ kip·in

$V_3 := -0.2$ kips $\quad M_3 := -85$ kip·in

$P_2 := |V_2|$

$M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

$P_3 := |V_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 9.13 \;\; in^2$ $\quad J := 0.54 \;\; in^4$ $\quad k_{nt3} := 0.75$

$A_e := A$ $\quad d := 8.00 \;\; in$ $\quad k_{nt2} := 0.71$

$I_3 := 110 \;\; in^4$ $\quad k := 0 + \frac{15}{16} \; in$ $\quad k_{lt3} := 0.75$ $\quad$ K values by LRFD Table C-C2.1

$I_2 := 37.1 \;\; in^4$ $\quad t_w := 0.285 \; in$ $\quad k_{lt2} := 0.71$

$S_3 := 27.5 in^3$ $\quad b_f := 7.995 \; in$ $\quad L_{b3} := 128 \;\; in$

$S_2 := 9.27 in^3$ $\quad t_f := 0.435 \;\; in$ $\quad L_{b2} := 128 \;\; in$

$Z_3 := 30.4 in^3$ $\quad C_w := 530 \;\; in^6$

$Z_2 := 14.1 in^3$

MATERIAL & CODE PROPERTIES

$E := 29000 ksi$ $\quad Fy := 36 \;\; ksi$ $\quad \phi_b := 0.90$ $\quad \phi_c := 0.85$

$G := 11200 ksi$ $\quad Fu := 58 \;\; ksi$ $\quad \phi_v := 0.90$ $\quad C_b := 1.0$

$Fr := 10 \quad ksi$ $\quad Fyf := Fy$ $\quad \phi_{ty} := 0.90$

$F_L := Fyf - Fr$ $\quad Fyw := Fy$ $\quad \phi_{tf} := 0.75$

C_b is conservatively taken equal to 1.0

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 2 \cdot k$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)

(eq. LRFD D1-2)

$\phi P_{nt} = 296 \qquad$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy - 10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2 \cdot t_f} \qquad \lambda_{flange} = 9.19 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \geq \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \leq 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 55.3 \qquad \lambda_{r\_web} = 90.8$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 21.5 \qquad \lambda_{web} < \lambda_{p\_web} \quad \text{OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right) \quad \text{(eq.LRFD E2-4)}$$

$$\lambda_c = 1$$

$$F_{cr} := \text{if}\left(\lambda_c \leq 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$

(eq.LRFD E2-2)

(eq.LRFD E2-3)

$$F_{cr} = 32.3 \quad \text{ksi}$$

$$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A \qquad \text{(eq.LRFD E2-1)}$$

$$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$$

$$\phi P_n = 251 \quad \text{kips}$$

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS	2 - 2 AXIS
$M_{p3\_initial} := Fy \cdot Z_3$	$M_{p2\_initial} := Fy \cdot Z_2$
$M_{y3} := Fy \cdot S_3$	$M_{y2} := Fy \cdot S_2$

$$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$$

$$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$$

$M_{p3} = 1094$ kip·in $\qquad$ $M_{p2} = 501$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS $\qquad$ 2 - 2 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}} \qquad \text{(eq. LRFD F1-4)}$$

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}} \qquad \text{(eq. LRFD F1-8)}$$

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2 \qquad \text{(eq. LRFD F1-9)}$$

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^{\,2}}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}}\right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 1094 \quad kip \cdot in$$

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} = 1058 \quad kip \cdot in$$

$$M_{n2} = 501 \quad kip \cdot in$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$

$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$

$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$

$M_{n3} = 1058$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$M_{nt} := 0$

$B1 := 1$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS	2 - 2 AXIS
$M_{lt3} := M_3$	$M_{lt2} := M_2$
$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$	$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$
$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\,2}}$	$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\,2}}$
$B2_3 := \frac{1}{1 - \frac{\lvert P_1 \rvert}{P_{e2\_3}}}$	$B2_2 := \frac{1}{1 - \frac{\lvert P_1 \rvert}{P_{e2\_2}}}$
$B2_3 = 1.05$	$B2_2 = 1.16$

$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$

$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$

$M_{u3} = 89.6$ kip·in $\qquad$ $M_{u2} = 18.5$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.819$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.702$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf\cdot A_f$$

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \text{if}(\lambda \leq r1, 0.6\cdot Fyw\cdot A_w, 0)$$

$$V_{n2} := \text{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \mathrm{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 150$ kips

$V_{n2} = 49.2$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3}$

$\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 135$ kips

$\phi V_{n2} = 44.3$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy \qquad \text{(eq. LRFD H2-2)}$$

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{A_f}$$

$f_{v3} = 0.0288$ ksi

$$\frac{f_{v3}}{F_v} = 0.00148$$

$$f_{v2} := \frac{P_2}{A_w}$$

$f_{v2} = 0.351$ ksi

$$\frac{f_{v2}}{F_v} = 0.018$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

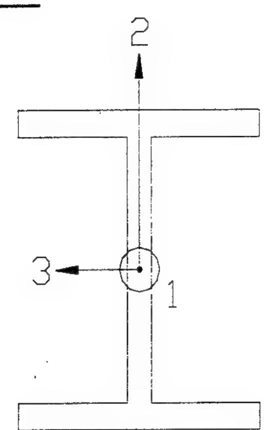
$\max(DCR) = 0.702$

APPENDIX D3

San Luis Obispo Shaft Intermediate Levels Beam (W16 x 40) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for I-shaped members
-Note member axis definitions below



the 1 axis is the longitudinal axis out of the page

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : W16x40 ; Beam at Intermediate Level

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 0$ kips $\qquad M_1 := 0$ kip·in

$V_2 := 10.7$ kips $\qquad M_2 := 0.0$ kip·in

$V_3 := 0.0$ kips $\qquad M_3 := -849$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$

$M_1 := |M_1| \qquad M_2 := |M_2| \qquad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 11.8 \quad in^2$ | $J := 0.79 \quad in^4$ | $k_{nt3} := 1.0$

$A_e := A$ | $d := 11.8 \quad in$ | $k_{nt2} := 1.0$

$I_3 := 518.0 \quad in^4$ | $k := 1 + \frac{3}{16} \quad in$ | $k_{lt3} := 1.0$ — K values by LRFD Table C-C2.1

$I_2 := 28.9 \quad in^4$ | $t_w := 0.305 \quad in$ | $k_{lt2} := 1.0$

$S_3 := 64.7 \quad in^3$ | $b_f := 6.995 \quad in$ | $L_{b3} := 90.0 \quad in$

$S_2 := 8.25 \quad in^3$ | $t_f := 0.505 \quad in$ | $L_{b2} := 199.0 \quad in$

$Z_3 := 72.9 \quad in^3$ | $C_w := 1730 \quad in^6$

$Z_2 := 12.7 \quad in^3$

MATERIAL & CODE PROPERTIES

$E := 29000 \, ksi$ | $Fy := 36 \quad ksi$ | $\phi_b := 0.90$ | $\phi_c := 0.85$

$G := 11200 \, ksi$ | $Fu := 58 \quad ksi$ | $\phi_v := 0.90$ | $C_b := 1.0$

$Fr := 10 \quad ksi$ | $Fyf := Fy$ | $\phi_{ty} := 0.90$ | C_b is conservatively taken equal to 1.0

$F_L := Fyf - Fr$ | $Fyw := Fy$ | $\phi_{tf} := 0.75$

CALCULATED PROPERTIES

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := d \cdot t_w$$

$$A_f := 2 \cdot b_f \cdot t_f$$

$$h := d - 2 \cdot k$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := if\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)
(eq. LRFD D1-2)

$\phi P_{nt} = 382 \quad$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy-10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2 \cdot t_f} \qquad \lambda_{flange} = 6.93 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \geq \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \leq 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 107 \qquad \lambda_{r\_web} = 162$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 30.9 \qquad \lambda_{web} < \lambda_{p\_web} \quad \text{OK}$$

NOTE : The width / thickness ratios must be less than the λr values above for the equations below to be valid. CHECK BY COMPARING THE NUMBERS!

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right) \qquad \text{(eq.LRFD E2-4)}$$

$$\lambda_c = 1.426$$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 15.4$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$

$\phi P_n = 382$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 2624$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 446$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS **2 - 2 AXIS**

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J} \right)^2$$ (eq. LRFD F1-9)

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^{\,2}}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}} \right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \mathrm{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 2112$ kip·in

$$M_{n3} := \mathrm{if}\left(L_{b2} < L_p, M_{p3}, 0 \right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \mathrm{if}\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := \mathrm{if}\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$M_{n3} = 2552$ kip·in $M_{n2} = 446$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \mathrm{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange} \right) \cdot \left(M_{nFLB} < M_{p3} \right), M_{nFLB}, M_{p3} \right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - (M_{p3} - M_r) \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \mathrm{if}\left[(\lambda_{web} > \lambda_{p\_web}) \cdot (M_{nWLB} < M_{p3}), M_{nWLB}, M_{p3} \right]$$

$$M_{n3} := \mathrm{if}(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3})$$

$$M_{n3} := \mathrm{if}(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3})$$

$M_{n3} = 2552$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\,2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\,2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 1$

$$M_{u3} := \mathrm{if}(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3})$$

$$M_{u2} := \mathrm{if}(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2})$$

$M_{u3} = 849$ kip·in

$M_{u2} = 0$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.37$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.333$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf \cdot A_f$$

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \mathrm{if}(\lambda \leq r1, 0.6\cdot Fyw \cdot A_w, 0)$$

$$V_{n2} := \mathrm{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \mathrm{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 153$ kips

$V_{n2} = 77.7$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3}$

$\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 137$ kips

$\phi V_{n2} = 70$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6 \cdot \phi_v \cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{2 \cdot A_f}$$

$$f_{v2} := \frac{P_2}{A_w}$$

$f_{v3} = 0$ ksi

$f_{v2} = 2.97$ ksi

$$\frac{f_{v3}}{F_v} = 0$$

$$\frac{f_{v2}}{F_v} = 0.153$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$\max(DCR) = 0.333$

APPENDIX D4

San Luis Obispo Shaft Brace-to-Beam Connection Evaluation

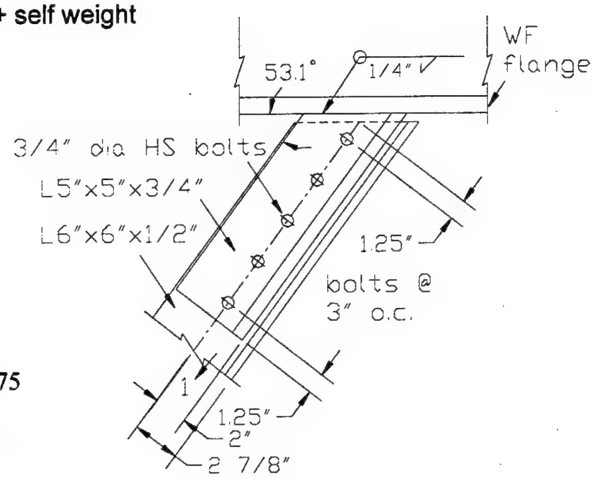
(based on AISC-LRFD, 2nd ed., 1994)

TOWER : Steel Braced Frame (San Luis Obispo, CA)

CONNETION : typical brace connection below (dwg. S-4)

ANALYSIS RUN : SLO 1, connection at foundation

APPLIED LOADS : 100% NEHRP-97 + self weight



MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 45.0$ kips

PROPERTIES:

$F_w := 70$	ksi	$\phi_v := 0.75$	
$F_b := 48$	ksi	$\phi := 0.75$	
$F_u := 58$	ksi	$\phi_y := 0.90$	
$F_y := 36$	ksi	$t_w := 0.25$	in

AXIAL CHECKS:

Weld of L5x5x3/4 to base plate:

$\text{throat} := 0.707 \cdot t_w$ $\text{throat} = 0.177$ in

$\text{length} := 2 \cdot 5 + 2 \cdot 4.25 + 2 \cdot 0.75$ $\text{length} = 20$ in

$R_{nw} := \text{throat} \cdot \text{length} \cdot 0.6 \cdot F_w$

$\phi_v \cdot R_{nw} = 111$ kips

Shear Strength of Bolts:

$d_b := 0.75$ in

$A_b := \frac{\pi \cdot d_b^{\,2}}{4}$

$R_{nbolt} := F_b \cdot A_b \cdot 5$

$\phi_v \cdot R_{nbolt} = 79.5$ kips

Bearing Strength at Bolt Holes:

$A_{bearing} := 2.4 \cdot d_b \cdot 0.5$

$R_{nbearing} := A_{bearing} \cdot F_u \cdot 5$

$\phi \cdot R_{nbearing} = 196$ kips

Strength of L5x5x3/4: (consider only leg attached to L6x6x1/2)

yield: $A_g := 5 \cdot 0.75$ in^2

$P_{ny} := F_y \cdot A_g$

$\phi_y \cdot P_{ny} = 122$ kips

fracture: $A_n := A_g - 0.75 \cdot (.75 + .125)$

$P_{nf} := F_u \cdot A_n$

$\phi \cdot P_{nf} = 135$ kips

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} \dfrac{P_1}{\phi_v \cdot R_{nw}} \\ \dfrac{P_1}{\phi_v \cdot R_{nbolt}} \\ \dfrac{P_1}{\phi_v \cdot R_{nbearing}} \\ \dfrac{P_1}{(\phi_y \cdot P_{ny})} \\ \dfrac{P_1}{(\phi_y \cdot P_{nf})} \end{bmatrix}$$

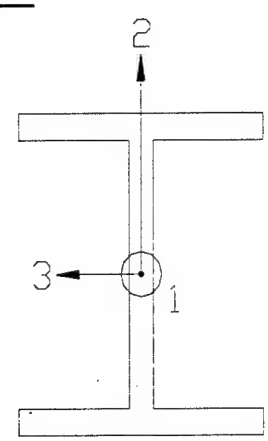
$\max(DCR) = 0.566$

APPENDIX D5

San Luis Obispo Shaft Junction Level Beam (W16 x 40) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for I-shaped members
-Note member axis definitions below



the 1 axis is the longitudinal axis out of the page

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : W16x40 ; beam at junction level

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 0.0$ kips $\quad M_1 := 0$ kip·in

$V_2 := -16.2$ kips $\quad M_2 := 0$ kip·in

$V_3 := 0.0$ kips $\quad M_3 := -825$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$

$M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 11.8\ in^2$ $J := 0.79\ in^4$ $k_{nt3} := 1.0$

$A_e := A$ $d := 11.8\ in$ $k_{nt2} := 1.0$

$I_3 := 518.0\ in^4$ $k := 1 + \frac{3}{16}\ in$ $k_{lt3} := 1.0$

$I_2 := 28.9\ in^4$ $t_w := 0.305\ in$ $k_{lt2} := 1.0$

K values by LRFD Table C-C2.1

$S_3 := 64.7\ in^3$ $b_f := 6.995\ in$ $L_{b3} := 90.0\ in$

$S_2 := 8.25\ in^3$ $t_f := 0.505\ in$ $L_{b2} := 199\ in$

$Z_3 := 72.9\ in^3$ $C_w := 1730\ in^6$

$Z_2 := 12.7\ in^3$

MATERIAL & CODE PROPERTIES

$E := 29000\ ksi$ $Fy := 36\ ksi$ $\phi_b := 0.90$ $\phi_c := 0.85$

$G := 11200\ ksi$ $Fu := 58\ ksi$ $\phi_v := 0.90$ $C_b := 1.0$

$Fr := 10\ ksi$ $Fyf := Fy$ $\phi_{ty} := 0.90$

$F_L := Fyf - Fr$ $Fyw := Fy$ $\phi_{tf} := 0.75$

C_b is conservatively taken equal to 1.0

CALCULATED PROPERTIES

$r_3 := \sqrt{\frac{I_3}{A}}$ $A_w := d \cdot t_w$

$A_f := 2 \cdot b_f \cdot t_f$

$h := d - 2 \cdot k$

$r_2 := \sqrt{\frac{I_2}{A}}$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \text{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)
(eq. LRFD D1-2)

$\phi P_{nt} = 382$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy - 10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2 \cdot t_f} \qquad \lambda_{flange} = 6.93 \qquad \lambda_{flange} < \lambda_{p\_flange} \text{ OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \geq \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \leq 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 107 \qquad \lambda_{r\_web} = 162$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 30.9 \qquad \lambda_{web} < \lambda_{p\_web} \text{ OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right) \qquad \text{(eq.LRFD E2-4)}$$

$$\lambda_c = 1.43$$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 15.4$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \text{if}(P_1 < 0, \phi P_{nc}, \phi P_{nt})$

$\phi P_n = 382$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS	2 - 2 AXIS
$M_{p3\_initial} := Fy \cdot Z_3$	$M_{p2\_initial} := Fy \cdot Z_2$
$M_{y3} := Fy \cdot S_3$	$M_{y2} := Fy \cdot S_2$

$M_{p3} := \text{if}(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3)$

$M_{p2} := \text{if}(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2)$

$M_{p3} = 2624$ kip·in

$M_{p2} = 446$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

$$M_r := F_L \cdot S_3$$ (eq. LRFD F1-7)

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^2}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - (M_{p3} - M_r)\left(\frac{L_{b3} - L_p}{L_r - L_p}\right)\right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS | 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}}\right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 2112$ kip·in

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3}\right]$$

$M_{n3} = 2552$ kip·in $M_{n2} = 446$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - (M_{p3} - M_r)\left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right] \quad \text{(eq. LRFD A-F1-3)}$$

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3} \right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$$M_{n3} = 2552 \quad \text{kip} \cdot \text{in}$$

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\,2}}$$

$$B2_3 := \frac{1}{1 - \frac{\left|P_1\right|}{P_{e2\_3}}}$$

$$B2_3 = 1$$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\,2}}$$

$$B2_2 := \frac{1}{1 - \frac{\left|P_1\right|}{P_{e2\_2}}}$$

$$B2_2 = 1$$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$$M_{u3} = 825 \quad \text{kip} \cdot \text{in}$$

$$M_{u2} = 0 \quad \text{kip} \cdot \text{in}$$

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := \left| P_1 \right|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.359$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.323$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf \cdot A_f$$

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \text{if}\left(\lambda \le r1, 0.6\cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \le \lambda)\cdot(\lambda \le r2), 0.6\cdot Fyw\cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n3} = 153$ kips

$\phi V_{n3} := \phi_v \cdot V_{n3}$

$\phi V_{n3} = 137$ kips

$V_{n2} = 77.7$ kips

$\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n2} = 70$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6 \cdot \phi_v \cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{2 \cdot A_f}$$

$f_{v3} = 0$ ksi

$$\frac{f_{v3}}{F_v} = 0$$

$$f_{v2} := \frac{P_2}{A_w}$$

$f_{v2} = 4.5$ ksi

$$\frac{f_{v2}}{F_v} = 0.232$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$\max(DCR) = 0.323$

APPENDIX D6

San Luis Obispo Brace Between Junction and Walkway (L6 x 6 x 1/2) Evaluation

(based on AISC-LRFD, 2nd ed. 1994, pp 6-277)

NOTES : -This sheet is for Single Angle bracing members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : L 6x6x1/2 ; Bracing between Junction Level and Walkway Level in compression

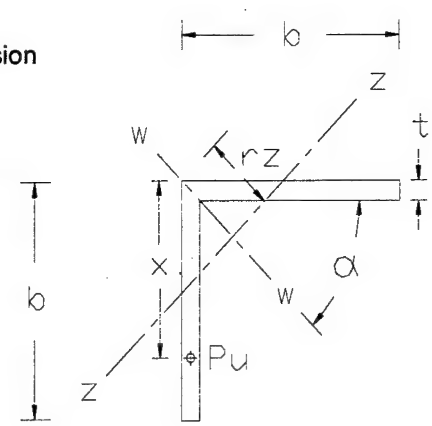
ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTION :

$P_u := 58.4$ kips

$P_u := |P_u|$



ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$b := 6.0$ in

$t := 0.5$ in

$A_g := 5.75$ in^2

$I_x := 19.9$ in^4

$I_y := I_x$

$r_z := 1.18$ in

$L := 8.62$ ft

$x := 2.875$ in

$y := 0.25$ in

$\alpha := 45.0$ deg

MATERIAL & CODE PROPERTIES

$F_y := 36$ ksi

$E := 29000\,\text{ksi}$

$k := 1.0$

$C_b := 1.0$

$C_m := 1.0$

$\phi := 0.9$

$\phi_b := 0.9$

CALCULATED PROPERTIES

$$\xi := x \cdot \sin(\alpha \cdot \deg) - y \cdot \cos(\alpha \cdot \deg) \qquad \xi = 1.86$$

$$\eta := x \cdot \cos(\alpha \cdot \deg) - \left[y \cdot \frac{(\cos(\alpha \cdot \deg))^2}{\sin(\alpha \cdot \deg)} \right] + \frac{y}{\sin(\alpha \cdot \deg)} \qquad \eta = 2.21$$

$w := \eta - r_z$ $\quad w = 1.03$ in $\quad$ distance from minor principal axis

$z := \xi$ $\quad z = 1.86$ in $\quad$ distance from major principal axis

$$I_z := A_g \cdot r_z^{\,2}$$

$$I_w := I_x + I_y - I_z$$

$$r_w := \sqrt{\frac{I_w}{A_g}} \qquad r_w = 2.35$$

AXIAL CHECKS

COMPRESSION

check if slender element, determine reduction factor , Q

$$Q := \text{if}\left[\frac{b}{t} > 0.446 \cdot \sqrt{\frac{E}{F_y}}, \left(1.34 - 0.761 \cdot \frac{b}{t} \cdot \sqrt{\frac{E}{F_y}}\right), 1.0\right]$$

$$Q := \text{if}\left[\frac{b}{t} > 0.910 \cdot \sqrt{\frac{E}{F_y}}, 0.534 \cdot \frac{E}{F_y \cdot \left(\frac{b}{t}\right)^2}, Q\right] \qquad Q = 1$$

$$\lambda_c := \frac{k \cdot L \cdot 12}{r_z \cdot \pi} \cdot \sqrt{\frac{F_y}{E}} \qquad \lambda_c = 0.983$$

$$F_{cr} := \text{if}\left[\lambda_c \cdot \sqrt{Q} \le 1.5, Q \cdot \left[\left(0.658^{Q \cdot \lambda_c^2}\right) \cdot F_y\right], \frac{0.877}{\lambda_c^2} \cdot F_y\right] \qquad F_{cr} = 24 \quad \text{ksi}$$

$P_n := A_g \cdot F_{cr}$ nominal strength for compression

$P_n = 138$ kips

BENDING CHECK

Bending about principle axes (5.3, pp. 6-285)

$$M_{oB} := \frac{C_b \cdot 0.46 \cdot E \cdot b^2 \cdot t^2}{L \cdot 12.0} \qquad M_{oB} = 1161 \quad \text{kip} \cdot \text{in}$$

$$C_w := b \cdot \sin(\alpha \cdot \deg)$$

$$M_y := F_y \cdot \frac{I_w}{C_w} \qquad M_y = 270 \quad \text{kip} \cdot \text{in}$$

5.1.3. Limit state of lateral-Torsional Buckling

$$M_{nwLTB} := \text{if}\left[M_{oB} \le M_y, \left(0.92 - 0.17 \cdot \frac{M_{oB}}{M_y}\right) \cdot M_{oB}, \left(1.58 - 0.83 \cdot \sqrt{\frac{M_y}{M_{oB}}}\right) \cdot M_y\right] \quad \begin{matrix}\text{(EQ 5-3a)} \\ \text{(EQ 5-3b)}\end{matrix}$$

$$M_{nwLTB} := \text{if}\left(M_{nwLTB} > 1.25 \cdot M_y, 1.25 \cdot M_y, M_{nwLTB}\right)$$

$M_{nwLTB} = 318$ kip·in

5.1.1. Limit State of Local Buckling

$$S_w := \frac{I_w}{C_w}$$

$$M_{nwLB} := \mathrm{if}\left[\frac{b}{t} > 0.382\cdot\sqrt{\frac{E}{F_y}}, F_y\cdot S_w\cdot\left[1.25 - 1.49\cdot\left[\frac{\frac{b}{t}}{0.382\cdot\sqrt{\frac{E}{F_y}}} - 1\right]\right], 1.25\cdot F_y\cdot S_w\right]$$

$$M_{nwLB} := \mathrm{if}\left(\frac{b}{t} > 0.446\cdot\sqrt{\frac{E}{F_y}}, Q\cdot F_y\cdot S_w, M_{nwLB}\right)$$

$M_{nwLB} = 294$ kip·in

$$M_{nw} := \mathrm{if}\left(M_{nwLB} < M_{nwLTB}, M_{nwLB}, M_{nwLTB}\right)$$

$M_{nw} = 294$ kip·in

Minor Principal Axis (z-z) Bending

$$C_{zTIP} := b\cdot\cos(\alpha\cdot deg) - \frac{t}{\sin(\alpha\cdot deg)}\cdot\left(\cos(\alpha\cdot deg)^2 - 1\right) - r_z$$

$C_{zTIP} = 3.42$

$$C_{zCORNER} := r_z$$

$$C_z := \mathrm{if}\left(C_{zTIP} > C_{zCORNER}, C_{zTIP}, C_{zCORNER}\right)$$

$$S_z := \frac{I_z}{C_z}$$

$$M_{nzLB} := \mathrm{if}\left[\frac{b}{t} > 0.382\cdot\sqrt{\frac{E}{F_y}}, F_y\cdot S_z\cdot\left[1.25 - 1.49\cdot\left[\frac{\frac{b}{t}}{0.382\cdot\sqrt{\frac{E}{F_y}}} - 1\right]\right], 1.25\cdot F_y\cdot S_z\right]$$

$$M_{nzLB} := \mathrm{if}\left(\frac{b}{t} > 0.446\cdot\sqrt{\frac{E}{F_y}}, Q\cdot F_y\cdot S_z, M_{nzLB}\right)$$

$M_{nzLB} = 92$ kip·in

$M_{nzy} := 1.25\cdot F_y\cdot S_z$ $\qquad M_{nzy} = 105$ kip·in

$M_{nz} := \mathrm{if}\left(w > 0, M_{nzLB}, M_{nzy}\right)$ $\qquad M_{nz} = 92$ kip·in

Determine Moment Amplification Factors

$$P_{e1w} := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot L \cdot 12.0}{r_w}\right)^2} \cdot A_g \qquad P_{e1w} = 850$$

$$B_{1w} := \frac{C_m}{1 - \frac{P_u}{P_{e1w}}} \qquad B_{1w} = 1.07$$

$$B_{1w} := \mathrm{if}\left(B_{1w} \ge 1.0, B_{1w}, 1.0\right)$$

$$M_{uw} := \left|B_{1w} \cdot z \cdot P_u\right| \qquad M_{uw} = 116 \quad \mathrm{kip \cdot in}$$

$$P_{e1z} := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot L \cdot 12.0}{r_z}\right)^2} \cdot A_g \qquad P_{e1z} = 214$$

$$B_{1z} := \frac{C_m}{1 - \frac{P_u}{P_{e1z}}} \qquad B_{1z} = 1.37$$

$$B_{1z} := \mathrm{if}\left(B_{1z} \ge 1.0, B_{1z}, 1.0\right)$$

$$M_{uz} := \left|B_{1z} \cdot w \cdot P_u\right| \qquad M_{uz} = 82.7 \quad \mathrm{kip \cdot in}$$

INTERACTION EQUATIONS

$$DCR := \mathrm{if}\left[\frac{P_u}{\phi \cdot P_n} \ge 0.2, \frac{P_u}{\phi \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_{uw}}{\phi_b \cdot M_{nw}} + \frac{M_{uz}}{\phi_b \cdot M_{nz}}\right), \frac{P_u}{2 \cdot \phi \cdot P_n} + \left(\frac{M_{uw}}{\phi_b \cdot M_{nw}} + \frac{M_{uz}}{\phi_b \cdot M_{nz}}\right)\right]$$

$DCR = 1.75$ includes resistance factors .

$$DCR := \mathrm{if}\left[\frac{P_u}{P_n} \ge 0.2, \frac{P_u}{P_n} + \frac{8}{9} \cdot \left(\frac{M_{uw}}{M_{nw}} + \frac{M_{uz}}{M_{nz}}\right), \frac{P_u}{2 \cdot P_n} + \left(\frac{M_{uw}}{M_{nw}} + \frac{M_{uz}}{M_{nz}}\right)\right]$$

$DCR = 1.57$ excludes resistance factors

APPENDIX D7

San Luis Obispo Beams at Base of Mullions (W8 x 35) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for I-shaped members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : W8x35 ; Beam at Base of Mullion

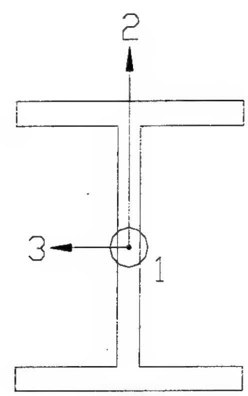
ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression



the 1 axis is the longitudinal axis out of the page

$P_1 := 9.2$ kips $\qquad M_1 := 1$ kip·in

$V_2 := 7.4$ kips $\qquad M_2 := -154$ kip·in

$V_3 := -1.9$ kips $\qquad M_3 := 663$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$

$M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 10.3 \ in^2$ | $J := 0.77 \ in^4$ | $k_{nt3} := 1.0$

$A_e := A$ | $d := 8.12 \ in$ | $k_{nt2} := 1.0$

$I_3 := 127. \ in^4$ | $k := 1 + \frac{0}{16} \ in$ | $k_{lt3} := 1.0$

$I_2 := 42.6 \ in^4$ | $t_w := 0.310 \ in$ | $k_{lt2} := 1.0$

K values by LRFD Table C-C2.1

$S_3 := 31.2 in^3$ | $b_f := 8.020 \ in$ | $L_{b3} := 120 \ in$

$S_2 := 10.6 in^3$ | $t_f := 0.495 \ in$ | $L_{b2} := 60 \ in$

$Z_3 := 34.7 in^3$ | $C_w := 619 \ in^6$

$Z_2 := 16.1 in^3$

MATERIAL & CODE PROPERTIES

$E := 29000 ksi$ | $Fy := 36 \ ksi$ | $\phi_b := 0.90$ | $\phi_c := 0.85$

$G := 11200 ksi$ | $Fu := 58 \ ksi$ | $\phi_v := 0.90$ | $C_b := 1.0$

$Fr := 10 \ ksi$ | $Fyf := Fy$ | $\phi_{ty} := 0.90$

C_b is conservatively taken equal to 1.0

$F_L := Fyf - Fr$ | $Fyw := Fy$ | $\phi_{tf} := 0.75$

CALCULATED PROPERTIES

$r_3 := \sqrt{\frac{I_3}{A}}$

$r_2 := \sqrt{\frac{I_2}{A}}$

$A_w := d \cdot t_w$

$A_f := 2 \cdot b_f \cdot t_f$

$h := d - 2 \cdot k$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := if\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)
(eq. LRFD D1-2)

$\phi P_{nt} = 334$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of I-shaped rolled beams and channels in flexure (for b/t ratio)

$$\lambda_{p\_flange} := \frac{65}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{141}{\sqrt{Fy-10}}$$

$$\lambda_{p\_flange} = 10.8 \qquad \lambda_{r\_flange} = 27.7$$

$$\lambda_{flange} := \frac{b_f}{2 \cdot t_f} \qquad \lambda_{flange} = 8.1 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := |P_1|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \text{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \text{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 98.6 \qquad \lambda_{r\_web} = 158$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 19.7 \qquad \lambda_{web} < \lambda_{p\_web} \quad \text{OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right) \quad \text{(eq.LRFD E2-4)}$$

$$\lambda_c = 0.383$$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right) \quad \text{(eq.LRFD E2-2)} \quad \text{(eq.LRFD E2-3)} \qquad F_{cr} = 33.9 \quad \text{ksi}$$

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \text{if}(P_1 < 0, \phi P_{nc}, \phi P_{nt})$

$\phi P_n = 334$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3)$

$M_{p3} = 1249$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2)$

$M_{p2} = 572$ kip·in

(a) : I-shaped members and channels

3 - 3 AXIS

$$L_p := \frac{300 \cdot r_2}{\sqrt{Fyf}}$$ (eq. LRFD F1-4)

$$X_1 := \frac{\pi}{S_3} \cdot \sqrt{\frac{E \cdot G \cdot J \cdot A}{2}}$$ (eq. LRFD F1-8)

$$X_2 := 4 \cdot \frac{C_w}{I_2} \cdot \left(\frac{S_3}{G \cdot J}\right)^2$$ (eq. LRFD F1-9)

$M_r := F_L \cdot S_3$ (eq. LRFD F1-7)

2 - 2 AXIS

$$L_r := \frac{r_2 \cdot X_1}{F_L} \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot F_L^{\,2}}}$$ (eq. LRFD F1-6)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS 2 - 2 AXIS

$$M_{nLTB\_elastic} := C_b \cdot \frac{\pi}{L_{b2}} \cdot \sqrt{E \cdot I_2 \cdot G \cdot J + \left(\frac{\pi \cdot E}{L_{b2}} \right)^2 \cdot I_2 \cdot C_w}$$ (eq. LRFD F1-13)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$M_{nLTB\_elastic} = 1249$ kip·in

$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0 \right)$ $M_{n2} := M_{p2}$

$M_{n3} := \text{if}\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$

$M_{n3} := \text{if}\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$

$M_{n3} = 1249$ kip·in $M_{n2} = 572$ kip·in

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange} \right) \cdot \left(M_{nFLB} < M_{p3} \right), M_{nFLB}, M_{p3} \right]$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right]$$ (eq. LRFD A-F1-3)

$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web} \right) \cdot \left(M_{nWLB} < M_{p3} \right), M_{nWLB}, M_{p3} \right]$

$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$

$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$

$M_{n3} = 1249$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$M_{nt} := 0$

$B1 := 1$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$M_{lt3} := M_3$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1$

2 - 2 AXIS

$M_{lt2} := M_2$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 1$

$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$

$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$

$M_{u3} = 663$ kip·in

$M_{u2} = 154$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.902$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.811$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u>

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

$V_{n3} = 171$ kips

$$\phi V_{n3} := \phi_v \cdot V_{n3}$$

<u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 69.7$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 87.2$$

$$V_{n2} := \text{if}\left(\lambda \leq r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6 \cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \leq \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$V_{n2} = 54.4$ kips

$$\phi V_{n2} := \phi_v \cdot V_{n2}$$

$\phi V_{n3} = 154$ kips $\phi V_{n2} = 48.9$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy$$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$f_{v3} := \frac{M_1}{A_f \cdot \frac{d}{2}} + \frac{P_3}{2 \cdot A_f} \qquad f_{v2} := \frac{P_2}{A_w}$$

$f_{v3} = 0.151$ ksi $f_{v2} = 2.94$ ksi

$$\frac{f_{v3}}{F_v} = 0.00775 \qquad \frac{f_{v2}}{F_v} = 0.151$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 0.811$$

APPENDIX D8

San Luis Obispo Cab Mullion to Beam (W10 x 26) Connection Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

TOWER : Steel Braced Frame (San Luis Obispo, CA)

CONNETION : Typ. moment connection at mullion base (dwg. S-3)

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :

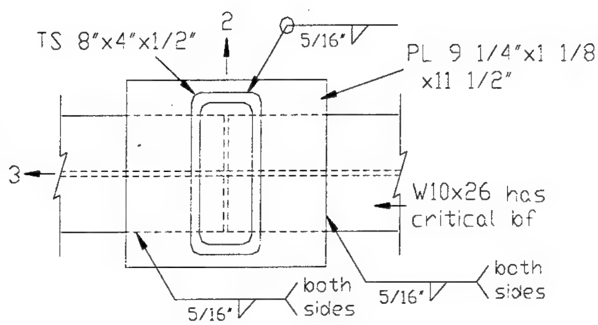
P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 5.2$ kips $M_1 := -49$ kip·in

$V_2 := -1.9$ kips $M_2 := -484$ kip·in

$V_3 := -14.3$ kips $M_3 := -121$ kip·in



The 1 axis is out of the page

$P_2 := |V_2|$

$P_3 := |V_3|$ $M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$

PROPERTIES

$TS_{flange} := 4$ in $leg := \frac{5}{16}$ in

$TS_{web} := 8$ in $throat := 0.707 \cdot leg$

$TS_{curvature} := 0.313$ in $L_{plate} := 11.5$ in

$b_f := 5.770$ in $t_{plate} := 1.125$ in

MATERIAL & CODE PROPERITIES

$F_w := 70$ ksi $\qquad \phi_v := 0.75$

$F_b := 48$ ksi $\qquad \phi := 0.75$

$F_u := 58$ ksi $\qquad \phi_y := 0.90$

$F_y := 36$ ksi

AXIAL CHECK:

Axial load on weld from mullion to base plate:

$$\phi R_n := \phi_v \cdot throat \cdot 0.6 \cdot F_w \cdot (2 \cdot TS_{web} + 2 \cdot TS_{flange}) \qquad \phi R_n = 167 \quad \text{kips}$$

BENDING CHECK:

Weld of mullion to base plate:

$$\phi M_{nx} := \phi_v \cdot throat \cdot 0.6 \cdot F_w \cdot [TS_{flange} \cdot (8 + leg) + TS_{web} \cdot 4] \qquad \phi M_{nx} = 454 \quad \text{kip} \cdot \text{in}$$

$$\phi M_{ny} := \phi_v \cdot throat \cdot 0.6 \cdot F_w \cdot [TS_{web} \cdot (4 + leg) + TS_{flange} \cdot 2] \qquad \phi M_{ny} = 296 \quad \text{kip} \cdot \text{in}$$

Weld of base plate to W10x26:

$$\phi M_{nx} := \phi_v \cdot throat \cdot 0.6 \cdot F_w \cdot \left(\frac{b_f^2}{2} + L_{plate} \cdot b_f \right) \qquad \phi M_{nx} = 578 \quad \text{kip} \cdot \text{in}$$

$$\phi M_{ny} := \left[\phi_v \cdot throat \cdot 0.6 \cdot F_w \cdot \left(L_{plate} \cdot b_f + \frac{L_{plate}^2}{2} \right) \right] \qquad \phi M_{ny} = 922 \quad \text{kip} \cdot \text{in}$$

Bending of base plate:

strong axis:

$$\phi M_n := \phi_y \cdot F_y \cdot \frac{t_{plate}^2}{4} \cdot 5$$

$$P_{u\_flange} := \frac{\phi M_n}{\left(\frac{8}{2} - \frac{1}{4} - \frac{5.770}{2} \right)} \qquad P_{u\_flange} = 59.3 \quad \text{kips}$$

$$\phi M_{nx} := P_{u\_flange} \cdot (TS_{web} - 0.5) \qquad \phi M_{nx} = 444 \quad \text{kip} \cdot \text{in}$$

weak axis:

$$\phi V_n := \phi_y \cdot 0.6 \cdot F_y \cdot \left(b_f + 1\right) \cdot t_{plate} \qquad \phi V_n = 148 \qquad \text{kips}$$

$$\phi M_{ny} := \phi V_n \cdot TS_{flange} \qquad \phi M_{ny} = 592 \qquad \text{kip} \cdot \text{in}$$

INTERACTION:

Weld from mullion to base plate controls the moment allowables:

$$\phi M_{nx} := 454 \qquad \text{kip in}$$

$$\phi M_{ny} := 296 \qquad \text{kip in}$$

$$DCR := \frac{P_1}{\phi R_n} + \frac{M_3}{\phi M_{nx}} + \frac{M_2}{\phi M_{ny}}$$

$$DCR = 1.93$$

APPENDIX D9

San Luis Obispo Mullion (TS 8 x 4 x 1/2) Above and Below Window Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

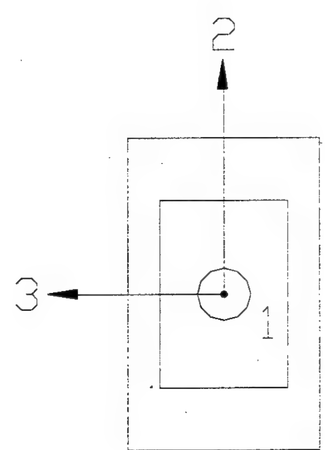
NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : TS 8x4x1/2 ; corner mullion

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight



MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -17.5$ kips $M_1 := -49$ kip·in

$V_2 := -1.9$ kips $M_2 := -484$ kip·in

$V_3 := -14.3$ kips $M_3 := -121$ kip·in

the 1 axis is the longitudinal axis out of the page

$P_2 := |V_2|$

$P_3 := |V_3|$ $M_1 := |M_1|$ $M_2 := |M_2|$ $M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 10.4 \quad in^2$

$I_3 := 75.1 \quad in^4$

$I_2 := 24.6 \quad in^4$

$Z_3 := 24.7 \quad in^3$

$Z_2 := 15.0 \quad in^3$

$J := 64.1 \quad in^4$

$d := 8 \quad in$

$t_w := 0.5 \quad in$

$b_f := 4.0 \quad in$

$t_f := 0.5 \quad in$

$k_{nt3} := 0.8$

$k_{nt2} := 0.8$

K values by LRFD
Table C-C2.1

$k_{lt3} := 2.10$

$k_{lt2} := 2.10$

$L_{b3} := 39.75 \quad in$

$L_{b2} := 30.00 \quad in$

MATERIAL & CODE PROPERTIES

$E := 29000\,ksi$

$G := 11200\,ksi$

$Fr := 10 \quad ksi$

$F_L := Fyf - Fr$

$Fy := 46 \quad ksi$

$Fu := 58 \quad ksi$

$Fyf := Fy$

$Fyw := Fy$

$\phi_b := 0.90$

$\phi_v := 0.90$

$\phi_{ty} := 0.90$

$\phi_{tf} := 0.75$

$\phi_c := 0.85$

$C_b := 1.0$

CALCULATED PROPERTIES

$A_e := A$

$$S_3 := \frac{I_3}{\left(\frac{d}{2}\right)} \qquad S_3 = 18.8 \quad in^3$$

$$S_2 := \frac{I_2}{\left(\frac{b_f}{2}\right)} \qquad S_2 = 12.3 \quad in^3$$

$$r_3 := \sqrt{\frac{I_3}{A}}$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

$A_w := 2 \cdot d \cdot t_w$

$A_f := 2 \cdot b_f \cdot t_f$

$h := d - 3 \cdot t_f$

$b := b_f - 3 \cdot t_w$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, \left(\phi_{tf}\cdot Fu\cdot A_e\right), \phi_{ty}\cdot Fy\cdot A\right]$$ (eq. LRFD D1-1) (eq. LRFD D1-2)

$\phi P_{nt} = 431$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of square and rectangular box and hollow structural sections (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$$

$$\lambda_{p\_flange} = 28 \qquad \lambda_{r\_flange} = 35.1$$

$$\lambda_{flange} := \frac{b}{t_f} \qquad \lambda_{flange} = 5 \qquad \lambda_{flange} < \lambda_{p\_flange} \text{ OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := \left|P_1\right|$$

$$P_y := Fy\cdot A$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left[\left(\frac{P_u}{\phi_b\cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b\cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75\cdot P_u}{\phi_b\cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74\cdot\frac{P_u}{\phi_b\cdot P_y}\right)$$

$$\lambda_{p\_web} = 83.8 \qquad \lambda_{r\_web} = 139$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 13 \qquad \lambda_{web} < \lambda_{p\_web} \text{ OK}$$

$$\lambda_c := \mathrm{if}\left(\frac{k_{lt3}\cdot L_{b3}}{r_3} > \frac{k_{lt2}\cdot L_{b2}}{r_2}, \frac{k_{lt3}\cdot L_{b3}}{r_3\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}, \frac{k_{lt2}\cdot L_{b2}}{r_2\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 0.519$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 41.1$ ksi

$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$ (eq.LRFD E2-1)

$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$

$\phi P_n = 363$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 1136$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 690$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-5)

$M_r := Fyf \cdot S_3$

2 - 2 AXIS

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A} \qquad \text{(eq. LRFD F1-10)}$$

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right] \qquad \text{(eq. LRFD F1-2)}$$

$$M_{nLTB\_inelastic} = 1146 \quad \text{kip} \cdot \text{in}$$

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2} \right)} \qquad \text{(eq. LRFD F1-14)}$$

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3}\right) \qquad \text{(eq. LRFD F1-12)}$$

$$M_{nLTB\_elastic} = 1136 \quad \text{kip} \cdot \text{in}$$

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0\right)$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2}\right) \cdot \left(L_{b2} \le L_r\right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2}\right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} = 1136 \quad \text{kip} \cdot \text{in}$$

2 - 2 AXIS

$$M_{n2} := M_{p2}$$

$$M_{n2} = 690 \quad \text{kip} \cdot \text{in}$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right] \qquad \text{(eq. LRFD A-F1-3)}$$

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right) \cdot \left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3} \right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right) \cdot \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right] \qquad \text{(eq. LRFD A-F1-3)}$$

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right) \cdot \left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3} \right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 1136$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\,2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1.01$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\,2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 1.01$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 122$ kip·in

$M_{u2} = 489$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := |P_1|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n}\ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b\cdot M_{n3}} + \frac{M_{u2}}{\phi_b\cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b\cdot M_{n3}} + \frac{M_{u2}}{\phi_b\cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 0.93$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n}\ge 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.836$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf\cdot A_f$$

$$\phi V_{n3} := \phi_v\cdot V_{n3}$$

$\phi V_{n3} = 99.4$ kips

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 61.6$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 77.1$$

$$V_{n2} := \text{if}\left(\lambda \le r1, 0.6\cdot Fyw\cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \le \lambda)\cdot(\lambda \le r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$$\phi V_{n2} := \phi_v\cdot V_{n2}$$

$\phi V_{n2} = 199$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$$F_v := 0.6 \cdot \phi_v \cdot Fy$$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$$A_o := (b_f - t_f) \cdot (d - t_w)$$

$$f_{v3} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_3}{2 \cdot A_f}$$

$$f_{v3} = 3.65 \quad \text{ksi}$$

$$\frac{f_{v3}}{F_v} = 0.147$$

$$f_{v2} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_2}{2 \cdot A_w}$$

$$f_{v2} = 1.99 \quad \text{ksi}$$

$$\frac{f_{v2}}{F_v} = 0.08$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$$\max(DCR) = 0.836$$

APPENDIX D10

San Luis Obispo Mullion (TS 8 x 4 x 1/2) Within Window Span Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

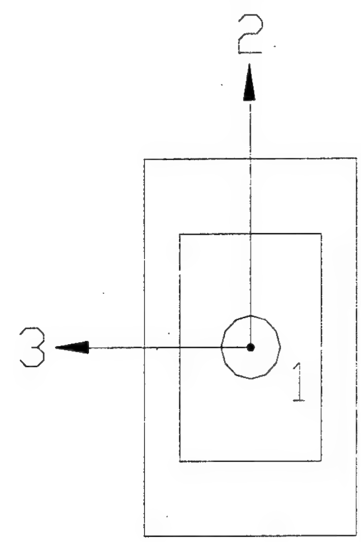
NOTES: -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : TS 8x4x1/2 ; corner mullion

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight



the 1 axis is the longitudinal axis out of the page

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -9.5$ kips $\qquad M_1 := 1$ kip·in

$V_2 := -3.8$ kips $\qquad M_2 := -307$ kip·in

$V_3 := -5.9$ kips $\qquad M_3 := -316$ kip·in

$P_2 := |V_2|$

$P_3 := |V_3|$ $\qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 10.4 \;\; in^2$ $\quad d := 8 \;\; in$ $\quad k_{nt3} := 0.8$

$I_3 := 75.1 \;\; in^4$ $\quad t_w := 0.5 \;\; in$ $\quad k_{nt2} := 0.8$

$I_2 := 24.6 \;\; in^4$ $\quad b_f := 4.0 \;\; in$ $\quad k_{lt3} := 2.10$

$Z_3 := 24.7\, in^3$ $\quad t_f := 0.5 \;\; in$ $\quad k_{lt2} := 2.10$

$Z_2 := 15.0\, in^3$ $\quad L_{b3} := 93.6 \;\; in$

$J := 64.1 \;\; in^4$ $\quad L_{b2} := 93.6 \;\; in$

K values by LRFD Table C-C2.1

MATERIAL & CODE PROPERTIES

$E := 29000\, ksi$ $\quad Fy := 46 \;\; ksi$ $\quad \phi_b := 0.90$ $\quad \phi_c := 0.85$

$G := 11200\, ksi$ $\quad Fu := 58 \;\; ksi$ $\quad \phi_v := 0.90$ $\quad C_b := 1.0$

$Fr := 10 \;\; ksi$ $\quad Fyf := Fy$ $\quad \phi_{ty} := 0.90$

$F_L := Fyf - Fr$ $\quad Fyw := Fy$ $\quad \phi_{tf} := 0.75$

CALCULATED PROPERTIES

$A_e := A$

$$S_3 := \frac{I_3}{\left(\frac{d}{2}\right)}$$

$$S_2 := \frac{I_2}{\left(\frac{b_f}{2}\right)}$$

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := 2 \cdot d \cdot t_w \qquad A_f := 2 \cdot b_f \cdot t_f$$

$$r_2 := \sqrt{\frac{I_2}{A}} \qquad h := d - 3 \cdot t_f \qquad b := b_f - 3 \cdot t_w$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, \left(\phi_{tf}\cdot Fu\cdot A_e\right), \phi_{ty}\cdot Fy\cdot A\right]$$

(eq. LRFD D1-1)

(eq. LRFD D1-2)

$\phi P_{nt} = 431$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of square and rectangular box and hollow structural sections (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$$

$$\lambda_{p\_flange} = 28 \qquad \lambda_{r\_flange} = 35.1$$

$$\lambda_{flange} := \frac{b}{t_f} \qquad \lambda_{flange} = 5 \qquad \lambda_{flange} < \lambda_{p\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := \left|P_1\right|$$

$$P_y := Fy\cdot A$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left[\left(\frac{P_u}{\phi_b\cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b\cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75\cdot P_u}{\phi_b\cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74\cdot\frac{P_u}{\phi_b\cdot P_y}\right)$$

$$\lambda_{p\_web} = 88.6 \qquad \lambda_{r\_web} = 141$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 13 \qquad \lambda_{web} < \lambda_{p\_web} \quad \text{OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3} \cdot L_{b3}}{r_3} > \frac{k_{lt2} \cdot L_{b2}}{r_2}, \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}, \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 1.62$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2} \cdot Fy, \frac{0.877}{\lambda_c^2} \cdot Fy\right)$$ (eq.LRFD E2-2)

(eq.LRFD E2-3)

$F_{cr} = 15.4$ ksi

$$\phi P_{nc} := \phi_c \cdot F_{cr} \cdot A$$ (eq.LRFD E2-1)

$$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$$

$\phi P_n = 136$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy \cdot Z_3$

$M_{y3} := Fy \cdot S_3$

$M_{p3} := \text{if}\left(Fy \cdot Z_3 \le 1.5 \cdot Fy \cdot S_3, Fy \cdot Z_3, 1.5 \cdot Fy \cdot S_3\right)$

$M_{p3} = 1136$ kip·in

2 - 2 AXIS

$M_{p2\_initial} := Fy \cdot Z_2$

$M_{y2} := Fy \cdot S_2$

$M_{p2} := \text{if}\left(Fy \cdot Z_2 \le 1.5 \cdot Fy \cdot S_2, Fy \cdot Z_2, 1.5 \cdot Fy \cdot S_2\right)$

$M_{p2} = 690$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-5)

$$M_r := Fyf \cdot S_3$$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-10)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

$$M_{nLTB\_inelastic} = 1140 \quad kip \cdot in$$

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS

2 - 2 AXIS

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2} \right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := \text{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 1136 \quad kip \cdot in$$

$$M_{n3} := \text{if}\left(L_{b2} < L_p, M_{p3}, 0 \right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := \text{if}\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := \text{if}\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} = 1136 \quad kip \cdot in$$

$$M_{n2} = 690 \quad kip \cdot in$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right)\cdot\left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right)\left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right)\cdot\left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 1136$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{2}}$$

$$B2_3 := \frac{1}{1 - \frac{\left|P_1\right|}{P_{e2\_3}}}$$

$B2_3 = 1.02$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{2}}$$

$$B2_2 := \frac{1}{1 - \frac{\left|P_1\right|}{P_{e2\_2}}}$$

$B2_2 = 1.05$

$$M_{u3} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 321$ kip·in $\qquad$ $M_{u2} = 324$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := P_1$$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) $\qquad$ (eq. LRFD H1-1b)

$C = 0.871$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2 \cdot \frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.782$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 61.6$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 77.1$$

$$V_{n2} := \mathrm{if}\left(\lambda \le r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \mathrm{if}\left[(r1 \le \lambda)\cdot(\lambda \le r2),\ 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)},\ V_{n2}\right]$$

$$V_{n2} := \mathrm{if}\left[r2 \le \lambda,\ \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2},\ V_{n2}\right]$$

$\phi V_{n3} := \phi_v\cdot V_{n3}$

$\phi V_{n3} = 99.4$ kips

$\phi V_{n2} := \phi_v\cdot V_{n2}$

$\phi V_{n2} = 199$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6\cdot\phi_v\cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$A_o := (b_f - t_f)\cdot(d - t_w)$

$$f_{v3} := \frac{M_1}{2\cdot t_f\cdot A_o} + \frac{P_3}{A_f}$$

$f_{v3} = 1.51$ ksi

$$\frac{f_{v3}}{F_v} = 0.0609$$

$$f_{v2} := \frac{M_1}{2\cdot t_f\cdot A_o} + \frac{P_2}{A_w}$$

$f_{v2} = 0.513$ ksi

$$\frac{f_{v2}}{F_v} = 0.0207$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$\max(DCR) = 0.782$

APPENDIX D11

San Luis Obispo Tubes Below Windows (TS 7 x 7 x 3/16) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

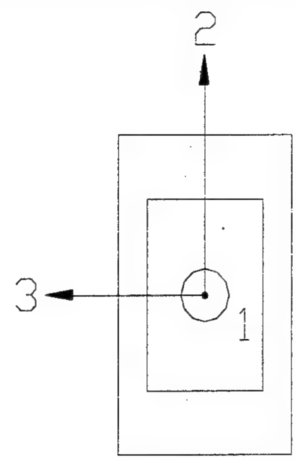
NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : TS 7x7x3/16 ; tubes at base of window

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight



the 1 axis is the longitudinal axis out of the page

MAXIMUM REACTIONS :

P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := -10.0\,\text{kips}$ $\qquad M_1 := 6\ \text{kip}\cdot\text{in}$

$V_2 := -13.7\,\text{kips}$ $\qquad M_2 := 145\ \text{kip}\cdot\text{in}$

$V_3 := -2.4\ \text{kips}$ $\qquad M_3 := 390\ \text{kip}\cdot\text{in}$

$P_2 := |V_2|$

$P_3 := |V_3|$ $\qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 5.02 \ in^2$ $d := 7.0 \ in$ $k_{nt3} := 0.65$

$I_3 := 38.5 \ in^4$ $t_w := 0.1875 \ in$ $k_{nt2} := 0.65$ K values by LRFD Table C-C2.1

$I_2 := 38.5 \ in^4$ $b_f := 7.0 \ in$ $k_{lt3} := 2.10$

$Z_3 := 12.7 \ in^3$ $t_f := 0.1875 \ in$ $k_{lt2} := 2.10$

$Z_2 := 12.7 \ in^3$ $L_{b3} := 115.0 \ in$

$J := 60.2 \ in^4$ $L_{b2} := 115.0 \ in$

MATERIAL & CODE PROPERTIES

$E := 29000 \ ksi$ $Fy := 46 \ ksi$ $\phi_b := 0.90$ $\phi_c := 0.85$

$G := 11200 \ ksi$ $Fu := 58 \ ksi$ $\phi_v := 0.90$ $C_b := 1.0$

$Fr := 10 \ ksi$ $Fyf := Fy$ $\phi_{ty} := 0.90$

$F_L := Fyf - Fr$ $Fyw := Fy$ $\phi_{tf} := 0.75$

CALCULATED PROPERTIES

$A_e := A$

$$S_3 := \frac{I_3}{\left(\frac{d}{2}\right)}$$

$$S_2 := \frac{I_2}{\left(\frac{b_f}{2}\right)}$$

$$r_3 := \sqrt{\frac{I_3}{A}} \qquad A_w := 2 \cdot d \cdot t_w \qquad A_f := 2 \cdot b_f \cdot t_f$$

$$r_2 := \sqrt{\frac{I_2}{A}} \qquad h := d - 3 \cdot t_f \qquad b := b_f - 3 \cdot t_w$$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf} \cdot Fu \cdot A_e \le \phi_{ty} \cdot Fy \cdot A, \left(\phi_{tf} \cdot Fu \cdot A_e\right), \phi_{ty} \cdot Fy \cdot A\right]$$

(eq. LRFD D1-1)

(eq. LRFD D1-2)

$\phi P_{nt} = 208$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of square and rectangular box and hollow structural sections (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$$

$$\lambda_{p\_flange} = 28 \qquad \lambda_{r\_flange} = 35.1$$

$$\lambda_{flange} := \frac{b}{t_f} \qquad \lambda_{flange} = 34.3 \qquad \lambda_{flange} < \lambda_{r\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := \left| P_1 \right|$$

$$P_y := Fy \cdot A$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left[\left(\frac{P_u}{\phi_b \cdot P_y} > 0.125\right) \cdot \left[\frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b \cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}} \cdot \left(1 - \frac{2.75 \cdot P_u}{\phi_b \cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}} \cdot \left(1 - 0.74 \cdot \frac{P_u}{\phi_b \cdot P_y}\right)$$

$$\lambda_{p\_web} = 81.9 \qquad \lambda_{r\_web} = 138$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 34.3 \qquad \lambda_{web} < \lambda_{r\_web} \quad \text{OK}$$

$$\lambda_c := \text{if}\left(\frac{k_{lt3}\cdot L_{b3}}{r_3} > \frac{k_{lt2}\cdot L_{b2}}{r_2}, \frac{k_{lt3}\cdot L_{b3}}{r_3\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}, \frac{k_{lt2}\cdot L_{b2}}{r_2\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$\lambda_c = 1.11$

$$F_{cr} := \text{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2}\cdot Fy, \frac{0.877}{\lambda_c^2}\cdot Fy\right)$$ (eq.LRFD E2-2)

(eq.LRFD E2-3)

$F_{cr} = 27.6$ ksi

$$\phi P_{nc} := \phi_c \cdot F_{cr}\cdot A$$

$$\phi P_n := \text{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$$ (eq.LRFD E2-1)

$\phi P_n = 118$

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$M_{p3\_initial} := Fy\cdot Z_3$

$M_{y3} := Fy\cdot S_3$

2 - 2 AXIS

$M_{p2\_initial} := Fy\cdot Z_2$

$M_{y2} := Fy\cdot S_2$

$$M_{p3} := \text{if}\left(Fy\cdot Z_3 \le 1.5\cdot Fy\cdot S_3, Fy\cdot Z_3, 1.5\cdot Fy\cdot S_3\right)$$

$$M_{p2} := \text{if}\left(Fy\cdot Z_2 \le 1.5\cdot Fy\cdot S_2, Fy\cdot Z_2, 1.5\cdot Fy\cdot S_2\right)$$

$M_{p3} = 584$ kip·in

$M_{p2} = 584$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{3750\cdot r_2}{M_{p3}}\cdot\sqrt{J\cdot A}$$ (eq. LRFD F1-5)

$$M_r := Fyf \cdot S_3$$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-10)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

$$M_{nLTB\_inelastic} = 587 \qquad kip \cdot in$$

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

<u>3 - 3 AXIS</u> <u>2 - 2 AXIS</u>

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2} \right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := \mathrm{if}\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 584 \qquad kip \cdot in$$

$$M_{n3} := \mathrm{if}\left(L_{b2} < L_p, M_{p3}, 0 \right) \qquad M_{n2} := M_{p2}$$

$$M_{n3} := \mathrm{if}\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := \mathrm{if}\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} = 584 \qquad kip \cdot in \qquad\qquad M_{n2} = 584 \qquad kip \cdot in$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \mathrm{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange} \right) \cdot \left(M_{nFLB} < M_{p3} \right), M_{nFLB}, M_{p3} \right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \mathrm{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right)\cdot\left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \mathrm{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \mathrm{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 514$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt}\cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3}\cdot L_{b3}}{r_3\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A\cdot Fy}{\lambda_{c3}^{\,2}}$$

$$B2_3 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_3}}}$$

$B2_3 = 1.06$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2}\cdot L_{b2}}{r_2\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A\cdot Fy}{\lambda_{c2}^{\,2}}$$

$$B2_2 := \frac{1}{1 - \frac{|P_1|}{P_{e2\_2}}}$$

$B2_2 = 1.06$

$$M_{u3} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1\cdot M_{nt} + B2_3\cdot M_{lt3}\right)$$

$$M_{u2} := \mathrm{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1\cdot M_{nt} + B2_2\cdot M_{lt2}\right)$$

$M_{u3} = 412$ kip·in

$M_{u2} = 153$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := \left| P_1 \right|$$

$$C := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2\cdot\phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) (eq. LRFD H1-1b)

$C = 1.22$ (including strength resistance factors)

$$C1 := \mathrm{if}\left[\frac{P_u}{\phi P_n} \geq 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9}\cdot\left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2\cdot\frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 1.1$ (excluding strength resistance factors)

<u>SHEAR</u> (LRFD F2, p. 6-56)

<u>3 - 3 AXIS</u>

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6\cdot Fyf \cdot A_f$$

<u>2 - 2 AXIS</u>

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 61.6$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 77.1$$

$$V_{n2} := \mathrm{if}\left(\lambda \leq r1, 0.6\cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \mathrm{if}\left[(r1 \leq \lambda)\cdot(\lambda \leq r2), 0.6\cdot Fyw \cdot A_w \cdot \frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000 \cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$\phi V_{n3} := \phi_v \cdot V_{n3}$ $\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 65.2$ kips $\phi V_{n2} = 65.2$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6 \cdot \phi_v \cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$A_o := (b_f - t_f) \cdot (d - t_w)$

$$f_{v3} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_3}{2 \cdot A_f} \qquad f_{v2} := \frac{M_1}{2 \cdot t_f \cdot A_o} + \frac{P_2}{2 \cdot A_w}$$

$f_{v3} = 0.802$ ksi $f_{v2} = 2.95$ ksi

$$\frac{f_{v3}}{F_v} = 0.0323 \qquad \frac{f_{v2}}{F_v} = 0.119$$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix} \qquad \max(DCR) = 1.1$$

APPENDIX D12

San Luis Obispo Tubes Above Windows (TS 7 x 7 x 3/16) Evaluation

(based on AISC-LRFD, 2nd ed., 1994)

NOTES : -This sheet is for Rectangular Structural tube members
-Note member axis definitions below

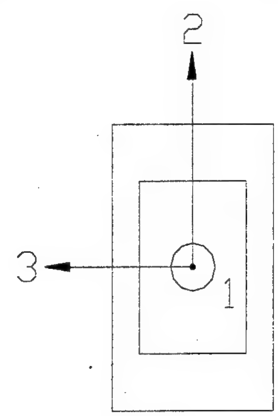
TOWER : Steel Braced Frame (San Luis Obispo, CA)

MEMBER : TS 7x7x3/16 ; tubes at top of window

ANALYSIS RUN : SLO 1

APPLIED LOADS : 100% NEHRP-97 + self weight

MAXIMUM REACTIONS :



P_1 (+), positive, represents tension

P_1 (-), negative, represents compression

$P_1 := 7.3$ kips $\quad M_1 := 1$ kip·in

$V_2 := -6.6$ kips $\quad M_2 := -74$ kip·in

$V_3 := 1.9$ kips $\quad M_3 := 244$ kip·in

the 1 axis is the longitudinal axis out of the page

$P_2 := |V_2|$

$P_3 := |V_3| \qquad M_1 := |M_1| \quad M_2 := |M_2| \quad M_3 := |M_3|$

ANALYSIS NOTES

-In considering second order effects of the building systems, the moment reactions above M1, M2, and M3 were computed including lateral translation of the frame joints. Moment reactions where the frame is restrained against lateral translations were not computed, and therefore have values of zero when the stresses are combined in the final section of this sheet.

SECTION PROPERTIES

$A := 5.02\ in^2$ $d := 7.0\ in$ $k_{nt3} := 0.65$

$I_3 := 38.5\ in^4$ $t_w := 0.1875\ in$ $k_{nt2} := 0.65$ K values by LRFD Table C-C2.1

$I_2 := 38.5\ in^4$ $b_f := 7.0\ in$ $k_{lt3} := 2.10$

$Z_3 := 12.7\,in^3$ $t_f := 0.1875\ in$ $k_{lt2} := 2.10$

$Z_2 := 12.7\,in^3$ $L_{b3} := 144.7\ in$

$J := 60.2\ in^4$ $L_{b2} := 144.7\ in$

MATERIAL & CODE PROPERTIES

$E := 29000\,ksi$ $Fy := 46\ ksi$ $\phi_b := 0.90$ $\phi_c := 0.85$

$G := 11200\,ksi$ $Fu := 58\ ksi$ $\phi_v := 0.90$ $C_b := 1.0$

$Fr := 10\ ksi$ $Fyf := Fy$ $\phi_{ty} := 0.90$

$F_L := Fyf - Fr$ $Fyw := Fy$ $\phi_{tf} := 0.75$

CALCULATED PROPERTIES

$$S_3 := \frac{I_3}{\left(\frac{d}{2}\right)}$$

$$S_2 := \frac{I_2}{\left(\frac{b_f}{2}\right)}$$

$$r_3 := \sqrt{\frac{I_3}{A}}$$

$$r_2 := \sqrt{\frac{I_2}{A}}$$

$A_e := A$

$A_w := 2 \cdot d \cdot t_w$

$A_f := 2 \cdot b_f \cdot t_f$

$h := d - 3 \cdot t_f$

$b := b_f - 3 \cdot t_w$

AXIAL CHECKS

TENSION (LRFD D, p. 6-44)

$$\phi P_{nt} := \mathrm{if}\left[\phi_{tf}\cdot Fu\cdot A_e \le \phi_{ty}\cdot Fy\cdot A, \left(\phi_{tf}\cdot Fu\cdot A_e\right), \phi_{ty}\cdot Fy\cdot A\right]$$

(eq. LRFD D1-1)

(eq. LRFD D1-2)

$\phi P_{nt} = 208$ kips

COMPRESSION (LRFD E, p. 6-47)

E2. Design compressive strength for flexural buckling

Limiting values for local buckling (LRFD Table B5.1)

Flanges of square and rectangular box and hollow structural sections (for b/t ratio)

$$\lambda_{p\_flange} := \frac{190}{\sqrt{Fy}} \qquad \lambda_{r\_flange} := \frac{238}{\sqrt{Fy}}$$

$$\lambda_{p\_flange} = 28 \qquad \lambda_{r\_flange} = 35.1$$

$$\lambda_{flange} := \frac{b}{t_f} \qquad \lambda_{flange} = 34.3 \qquad \lambda_{flange} < \lambda_{r\_flange} \quad \text{OK}$$

Webs in combined flexure and compression (for h/t ratio)

$$P_u := \left|P_1\right|$$

$$P_y := Fy\cdot A$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left[\left(\frac{P_u}{\phi_b\cdot P_y} > 0.125\right)\cdot\left[\frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right) \ge \frac{253}{\sqrt{Fy}}\right]\right], \frac{191}{\sqrt{Fy}}\cdot\left(2.33 - \frac{P_u}{\phi_b\cdot P_y}\right), \frac{253}{\sqrt{Fy}}\right]$$

$$\lambda_{p\_web} := \mathrm{if}\left[\left(\frac{P_u}{\phi_b\cdot P_y} \le 0.125\right), \frac{640}{\sqrt{Fy}}\cdot\left(1 - \frac{2.75\cdot P_u}{\phi_b\cdot P_y}\right), \lambda_{p\_web}\right]$$

$$\lambda_{r\_web} := \frac{970}{\sqrt{Fy}}\cdot\left(1 - 0.74\cdot\frac{P_u}{\phi_b\cdot P_y}\right)$$

$$\lambda_{p\_web} = 85.2 \qquad \lambda_{r\_web} = 139$$

member values

$$\lambda_{web} := \frac{h}{t_w} \qquad \lambda_{web} = 34.3 \qquad \lambda_{web} < \lambda_{r\_web} \quad \text{OK}$$

$$\lambda_c := \mathrm{if}\left(\frac{k_{lt3}\cdot L_{b3}}{r_3} > \frac{k_{lt2}\cdot L_{b2}}{r_2}, \frac{k_{lt3}\cdot L_{b3}}{r_3\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}, \frac{k_{lt2}\cdot L_{b2}}{r_2\cdot\pi}\cdot\sqrt{\frac{Fy}{E}}\right)$$ (eq.LRFD E2-4)

$$\lambda_c = 1.39$$

$$F_{cr} := \mathrm{if}\left(\lambda_c \le 1.5, 0.658^{\lambda_c^2}\cdot Fy, \frac{0.877}{\lambda_c^2}\cdot Fy\right)$$ (eq.LRFD E2-2) (eq.LRFD E2-3)

$F_{cr} = 20.5$ ksi

$$\phi P_{nc} := \phi_c\cdot F_{cr}\cdot A$$ (eq.LRFD E2-1)

$$\phi P_n := \mathrm{if}\left(P_1 < 0, \phi P_{nc}, \phi P_{nt}\right)$$

$\phi P_n = 208$ kips

BEAM CHECK (ch. F, p. 6-52)

F1.2a. Doubly Symmetric Shapes and Channels for Lb<Lr

Lb = laterally unbraced length (defined above for the 3-3 and 2-2 axes)
Lr = limiting laterally unbraced length for lateral-torsional buckling

The flexure equations in section 2a are for Lb<Lr. In the majority of the members in the structures, probably Lb<Lr due to the above slabs and roof diaphragms. For columns undergoing sidesway, Lr may be less than Lb (section 2b.). In all cases, the lateral bracing effects should be investigated in the structural drawings.

3 - 3 AXIS

$$M_{p3\_initial} := Fy\cdot Z_3$$

$$M_{y3} := Fy\cdot S_3$$

2 - 2 AXIS

$$M_{p2\_initial} := Fy\cdot Z_2$$

$$M_{y2} := Fy\cdot S_2$$

$$M_{p3} := \mathrm{if}\left(Fy\cdot Z_3 \le 1.5\cdot Fy\cdot S_3, Fy\cdot Z_3, 1.5\cdot Fy\cdot S_3\right)$$

$$M_{p2} := \mathrm{if}\left(Fy\cdot Z_2 \le 1.5\cdot Fy\cdot S_2, Fy\cdot Z_2, 1.5\cdot Fy\cdot S_2\right)$$

$M_{p3} = 584$ kip·in

$M_{p2} = 584$ kip·in

(a) : rectangular bars and box sections

3 - 3 AXIS

2 - 2 AXIS

$$L_p := \frac{3750 \cdot r_2}{M_{p3}} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-5)

$$M_r := Fyf \cdot S_3$$

$$L_r := \frac{57000 \cdot r_2}{M_r} \cdot \sqrt{J \cdot A}$$ (eq. LRFD F1-10)

$$M_{nLTB\_inelastic} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{L_{b3} - L_p}{L_r - L_p} \right) \right]$$ (eq. LRFD F1-2)

$$M_{nLTB\_inelastic} = 587 \quad kip \cdot in$$

2b. Doubly Symmetric Shapes and Channels with Lb>Lr

(a) : I-shaped members and channels

3 - 3 AXIS

2 - 2 AXIS

$$M_{nLTB\_elastic} := \frac{57000 \cdot C_b \cdot \sqrt{J \cdot A}}{\left(\frac{L_{b2}}{r_2} \right)}$$ (eq. LRFD F1-14)

$$M_{nLTB\_elastic} := if\left(M_{nLTB\_elastic} \le M_{p3}, M_{nLTB\_elastic}, M_{p3} \right)$$ (eq. LRFD F1-12)

$$M_{nLTB\_elastic} = 584 \quad kip \cdot in$$

$$M_{n3} := if\left(L_{b2} < L_p, M_{p3}, 0 \right)$$

$$M_{n2} := M_{p2}$$

$$M_{n3} := if\left[\left(L_p \le L_{b2} \right) \cdot \left(L_{b2} \le L_r \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} := if\left[\left(L_r \le L_{b2} \right), M_{nLTB\_inelastic}, M_{n3} \right]$$

$$M_{n3} = 584 \quad kip \cdot in$$

$$M_{n2} = 584 \quad kip \cdot in$$

Check local limit states: (LRFD Appendix F, p. 6-111)

$$M_{nFLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r \right) \cdot \left(\frac{\lambda_{flange} - \lambda_{p\_flange}}{\lambda_{r\_flange} - \lambda_{p\_flange}} \right) \right]$$ (eq. LRFD A-F1-3)

$$M_{nFLB} := \text{if}\left[\left(\lambda_{flange} > \lambda_{p\_flange}\right)\cdot\left(M_{nFLB} < M_{p3}\right), M_{nFLB}, M_{p3}\right]$$

$$M_{nWLB} := C_b \cdot \left[M_{p3} - \left(M_{p3} - M_r\right)\left(\frac{\lambda_{web} - \lambda_{p\_web}}{\lambda_{r\_web} - \lambda_{p\_web}}\right)\right]$$ (eq. LRFD A-F1-3)

$$M_{nWLB} := \text{if}\left[\left(\lambda_{web} > \lambda_{p\_web}\right)\cdot\left(M_{nWLB} < M_{p3}\right), M_{nWLB}, M_{p3}\right]$$

$$M_{n3} := \text{if}\left(M_{nFLB} < M_{n3}, M_{nFLB}, M_{n3}\right)$$

$$M_{n3} := \text{if}\left(M_{nWLB} < M_{n3}, M_{nWLB}, M_{n3}\right)$$

$M_{n3} = 514$ kip·in

INTERACTION EQUATIONS

Second Order Effects: (LRFD C1)

$$M_{nt} := 0$$

$$B1 := 1$$

NOTE: As the product of $M_{nt} \cdot B1$ is zero, B1 was set to 1 for simplicity.

3 - 3 AXIS

$$M_{lt3} := M_3$$

$$\lambda_{c3} := \frac{k_{lt3} \cdot L_{b3}}{r_3 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_3} := \frac{A \cdot Fy}{\lambda_{c3}^{\,2}}$$

$$B2_3 := \frac{1}{1 - \frac{\left|P_1\right|}{P_{e2\_3}}}$$

$B2_3 = 1.07$

2 - 2 AXIS

$$M_{lt2} := M_2$$

$$\lambda_{c2} := \frac{k_{lt2} \cdot L_{b2}}{r_2 \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$

$$P_{e2\_2} := \frac{A \cdot Fy}{\lambda_{c2}^{\,2}}$$

$$B2_2 := \frac{1}{1 - \frac{\left|P_1\right|}{P_{e2\_2}}}$$

$B2_2 = 1.07$

$$M_{u3} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt3}, B1 \cdot M_{nt} + B2_3 \cdot M_{lt3}\right)$$

$$M_{u2} := \text{if}\left(P_1 > 0, M_{nt} + M_{lt2}, B1 \cdot M_{nt} + B2_2 \cdot M_{lt2}\right)$$

$M_{u3} = 244$ kip·in $\qquad$ $M_{u2} = 74$ kip·in

Symmetric Members Subject to Bending and Axial Forces (LRFD H1.1,2 p. 6-59)

let C = combination factor

NOTE: C should be less than or equal to 1.0

$$P_u := \left| P_1 \right|$$

$$C := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\phi P_n} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right), \frac{P_u}{2 \cdot \phi P_n} + \left(\frac{M_{u3}}{\phi_b \cdot M_{n3}} + \frac{M_{u2}}{\phi_b \cdot M_{n2}}\right)\right]$$

(eq. LRFD H1-1a) $\qquad$ (eq. LRFD H1-1b)

$C = 0.685$ (including strength resistance factors)

$$C1 := \text{if}\left[\frac{P_u}{\phi P_n} \ge 0.2, \frac{P_u}{\frac{\phi P_n}{0.85}} + \frac{8}{9} \cdot \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right), \frac{P_u}{2 \cdot \frac{\phi P_n}{0.85}} + \left(\frac{M_{u3}}{M_{n3}} + \frac{M_{u2}}{M_{n2}}\right)\right]$$

$C1 = 0.616$ (excluding strength resistance factors)

SHEAR (LRFD F2, p. 6-56)

3 - 3 AXIS

Flanges will not buckle due to shear as in the web. Therefore:

$$V_{n3} := 0.6 \cdot Fyf \cdot A_f$$

2 - 2 AXIS

$$\lambda := \frac{h}{t_w}$$

$$r1 := \frac{418}{\sqrt{Fyw}} \qquad r1 = 61.6$$

$$r2 := \frac{523}{\sqrt{Fyw}} \qquad r2 = 77.1$$

$$V_{n2} := \text{if}\left(\lambda \le r1, 0.6 \cdot Fyw \cdot A_w, 0\right)$$

$$V_{n2} := \text{if}\left[(r1 \le \lambda)\cdot(\lambda \le r2), 0.6\cdot Fyw\cdot A_w\cdot\frac{\frac{418}{\sqrt{Fyw}}}{\left(\frac{h}{t_w}\right)}, V_{n2}\right]$$

$$V_{n2} := \text{if}\left[r2 \le \lambda, \frac{132000\cdot A_w}{\left(\frac{h}{t_w}\right)^2}, V_{n2}\right]$$

$\phi V_{n3} := \phi_v \cdot V_{n3}$ $\phi V_{n2} := \phi_v \cdot V_{n2}$

$\phi V_{n3} = 65.2$ kips $\phi V_{n2} = 65.2$ kips

Unsymmetric members and members under torsion and combined torsion, flexure, shear, and / or axial force (LRFD H2, p. 6-60)

Limiting Shear Stress Value

$F_v := 0.6\cdot\phi_v\cdot Fy$ (eq. LRFD H2-2)

Combined Shear and Torsion stress:

$A_o := (b_f - t_f)\cdot(d - t_w)$

$$f_{v3} := \frac{M_1}{2\cdot t_f\cdot A_o} + \frac{P_3}{A_f} \qquad f_{v2} := \frac{M_1}{2\cdot t_f\cdot A_o} + \frac{P_2}{A_w}$$

$f_{v3} = 0.781$ ksi $f_{v2} = 2.57$ ksi

$\frac{f_{v3}}{F_v} = 0.0315$ $\frac{f_{v2}}{F_v} = 0.104$

DEMAND CAPACITY RATIO (DCR)

$$DCR := \begin{bmatrix} C1 \\ \frac{f_{v3}}{F_v} \\ \frac{f_{v2}}{F_v} \end{bmatrix}$$

$\max(DCR) = 0.616$

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